VOL. LIII. OCTOBER, 1927. No. 8.

REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

This Society is not responsible for any statement made or opinion expressed in its publications.

CONTENTS

For Index to all Papers, the discussion of which is current in *Proceedings*, see the second page of the cover.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

NEW THEORY FOR THE CENTRIFUGAL PUMP

By A. F. Sherzer,* Assoc. M. Am. Soc. C. E.

To Be Presented October 5, 1927

Synopsis

This paper presents an outline of a program of research and experiment on centrifugal pumps conducted in the Hydraulic Laboratory of the University of Michigan. This study has resulted in:

1.—The discovery of a number of errors in present accepted theories;

2.—The development of an entirely new theory; and

3.—The successful application of the new theory in increasing the efficiency of the centrifugal pump.

Introduction

It is a matter of common knowledge among those intimately connected with the design and manufacture of the centrifugal pump that most of the literature on this subject is of little assistance in their work. The actual design of a centrifugal pump is based largely on the known performance of similar pumps, and the designer determines empirically the various dimensions, constants, etc. An English writer on hydraulics, John Perry, makes the following significant statement:

"In this subject of flowing water there are more misleading hypotheses, due to perverted ingenuity, than in almost any other; and unfortunately the logical conclusions drawn from these hypotheses when known to be untrue are said to be the statements of theory as opposed to practice."

A careful examination of the centrifugal pump theory will show, it is believed, that he has hit the nail squarely on the head.

Note.—This paper is issued before the date set for presentation and discussion. Correspondence is invited and may be sent by mail to the Secretary. Discussion on the paper will be closed in February, 1928.

* Associate Prof., Mechanical Eng., Univ. of Michigan; Cons. Hydro-Mech. Engr., Ann Arbor, Mich.

Notation

In this paper, the following symbols are used:

1 or e = point on entrance edge of runner vane.
2 or a = point on discharge edges of runner vane.
r_1 = radial distance to Point 1.
r_2 = radial distance to Point 2.
V_2 or U_a = peripheral speed of the runner at its outer diameter.
V_1 or U_e = peripheral speed of the runner at its inner diameter.
C_2 or W_a = absolute velocity of the water discharged from the runner.
C_1 or W_e = absolute velocity of the water at entrance to the runner.
W_2 or V_a = relative velocity of the water to the runner at discharge.
W_1 or V_e = relative velocity of the water to the runner at entrance.
g = acceleration due to gravity.
γ = weight per unit volume of the liquid, in pounds.
H_{at} = height of water column supported by atmospheric pressure.
D_3 = outer diameter of the vortex formed in the pump casing.
V_3 = peripheral speed of outer diameter of the vortex.
C_{p2} = tangential component of C_2.
C_{p1} = " " " C_1.
α_2 or δ_a = angle between C_2 and V_2.
α_1 or δ_e = " " C_1 and V_1.
β_2 or β_a = " " W_2 and V_2.
β_1 or β_e = " " W_1 and V_1.
ω = angular velocity, in radians per second.
A_1, A_2, A_3 = areas of Orifices Nos. 1, 2, and 3. (See Fig. 8.)
H_1, H_2, H_3 = decreases in pressure head due to flow through Orifices Nos. 1, 2, and 3.

The Present Theory

From among many references setting forth the usual theory of the centrifugal pump, three are selected as typical:

(1) "Pumping Machinery", by A. M. Greene, Jr. His treatment is similar to "Die Centrifugal Pumpen", by Neuman, and is typical of the German method.

(2) "Centrifugal Pumps", by Lowenstein and Crissey.

(3) "Centrifugal Pumps", by R. L. Daugherty. This derivation of theory resembles that found in British texts.

A review of these and similar works indicates that the theoretical working equations may be derived, in general, from two distinct lines of approach. One of these seems to be held in especial favor by European writers, particularly in Germany, and is based on the application of Bernoulli's theorem to the flow through the pump. The other is often found in works of British and American origin and is based on the laws of angular momentum.

Present literature on this subject as typified by these authorities appears to be both hydraulically and mathematically incorrect. Further, the fundamental laws of action of the centrifugal pump seems never to have been

properly explained, and many misconceptions exist, even among those supposedly familiar with the hydraulics of centrifugal pumps.

Derivation by Application of Bernoulli's Theorem.—To simplify matters, the discussion will be confined to the derivation of Lowenstein and Crissey.* This corresponds closely with that given by Greene, and the same remarks may usually be applied to both.†

This derivation contains a most important and interesting error which contributes largely to the defects in present pump theory. One derivation reads, as follows:

$$h_a + \frac{W a^2}{2 g} = h_l + \frac{W_l^2}{2 g} + p_l\, h_n \dots\dots\dots\dots\dots\dots (A)$$

This is merely the application of Bernoulli's theorem to the problem, since h_a is the pressure head in the fluid at Point *a,* and W_a is the absolute velocity of the water discharge from the impeller at Point *a*; while h_l is the pressure head at *l,* the entrance to the guide-vanes (a short distance from Point *a*), and $\frac{W_l^2}{2\, g}$ is the absolute velocity head at *l*. The term, $p_l\, h_n$, represents the hydraulic losses occasioned by the flow from Points *a* to *l*. This expression is Bernoulli's theorem in its usual form.

It is commonly considered that the total head at *a* is composed of a pressure head, h_a, and, in addition, a velocity head, $\frac{W_a^2}{2\, g}$. When the pump discharge is approximately zero, the pressure head, h_a, is very nearly $\frac{U_a^2}{2\, g}$ as is well known. Under the same conditions the relative velocity, V_a, becomes practically zero and, hence, the absolute velocity, W_a, approaches the peripheral speed of the impeller at *a* or U_a. The velocity head is, therefore, $\frac{U_a^2}{2\, g}$, or nearly so. As will be shown, this theory would logically lead to a discrepancy between theory and practice of about 100 per cent.

Current Fallacies.—The usual explanation of this discrepancy indicates a serious misconception not only of the action of the centrifugal pump, but of one of the fundamental laws of hydraulics. The error is due to a misapplication of the well-known Bernoulli theorem.

When water flows in a pipe it is customary to say that the total head is obtained by measuring the pressure head by a piezometer at right angles to the flow, and adding the velocity head. This is a correct application of the theorem. The pressure and velocity exist as separate and distinct factors. However, water being set in rotation in a cylindrical vessel by the action of a paddle or impeller, there is a pressure created by centrifugal force which is equal to $\frac{U_a^2}{2\, g}$ and which is part and parcel of the velocity itself. The total

* "Centrifugal Pumps," p. 7.

† See Appendix II for derivation in full.

head may be regarded as either, but not both. The pressure exists merely because of the velocity in the circular path, but certainly not in addition to it.

It would be as logical to say that the "hammer", swung around by the athlete, has a total energy equal to the sum of the kinetic energy of the ball and the tension in the wire. Clearly the tension is not energy, but merely a force existing because of the tendency of the ball to fly off on a tangent. The energy is entirely kinetic. The act of letting go of the wire (destroying the tension) does not change any of the original energy, and the hammer flies off at a tangent with a velocity, U_a, which is the same as its former peripheral velocity. Therefore, the total energy is $\frac{U_a^2}{2g}$.

It is clear that the pump gives a head of only $\frac{U_a^2}{2g}$ at "shut off", instead of $\frac{U_a^2}{g}$ as required by all present theories, for the very simple reason that the energy imparted is only $\frac{U_a^2}{2g}$ and naturally the water can have no more energy than is given it by the impeller.

One of the reasons that might be advanced as to the cause of this error in the pump theory is the widespread failure to distinguish properly between pressure and energy. Pressure may measure potential energy and may serve to transmit energy, but it is not in itself energy any more than force is work.

Derivation by Principle of Angular Momentum.—In order better to understand this derivation, refer to Fig. 1. The value, $C_2 \cos \alpha_2$, represents the tangential component of the absolute velocity leaving the impeller, and $\frac{1}{g} C_2 \cos \alpha_2$ is the tangential momentum per second for each pound of water passing through the impeller. Momentum per second is force, and, therefore, $\frac{1}{g} C_2 \cos \alpha_2$ represents a force acting on the impeller in a direction opposite that of rotation. This force may be conveniently regarded as concentrated at Point 2. The moment of this force is $\frac{1}{g} C_2 \cos \alpha_2 \, r_2$, in which, r_2 is the radial distance from the center of the shaft to Point 2. In a similar manner, $\frac{1}{g} C_1 \cos \alpha_1$ represents a force concentrated at Point 1 which, if α_1 is less than 90°, will assist in rotating the impeller. The moment of this force is $\frac{1}{g} C_1 \cos \alpha_1 \, r_1$. The total turning moment on the wheel is the difference of these two moments, or,

$$\frac{1}{g} C_2 \cos \alpha_2 \, r_2 - \frac{1}{g} C_1 \cos \alpha_1 \, r_1,$$

or,

$$\frac{1}{g}\,(C_2 \cos \alpha_2\, r_2 - C_1 \cos \alpha_1\, r_1)$$

If this torque be multiplied by the angular velocity, ω, the result is the work done by the impeller on the water, or,

$$\text{Work} = \frac{1}{g}\,(C_2 \cos \alpha_2\, r_2\, \omega - C_1 \cos \alpha_1\, r_1\, \omega)$$

but $\omega\, r_2 = V_2$ and $\omega\, r_1 = V_1$. Hence,

$$\text{Work, or head} = \frac{1}{g}\,(C_2\, V_2 \cos \alpha_2 - C_1\, V_1 \cos \alpha_1) \ldots\ldots\ldots\ldots (B)$$

This is exactly the equation obtained by the application of Bernoulli's theorem to the flow of water through the pump.*

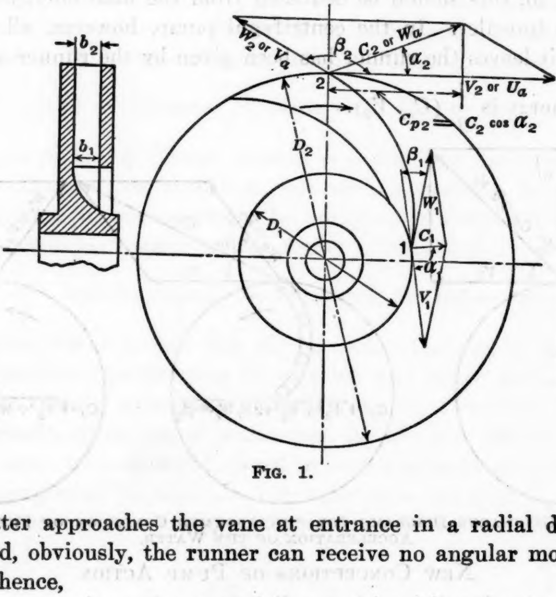


FIG. 1.

If the water approaches the vane at entrance in a radial direction, as is often assumed, obviously, the runner can receive no angular momentum from that source; hence,

$$C_p\, r_1 = 0$$

and,

$$H = \frac{1}{g}\left(C_{p2}\, r_2\right) = \frac{1}{g}\left(C_2\, V_2 \cos \alpha_2\right)^{\dagger} \ldots\ldots\ldots\ldots (C)$$

Evidently, as far as the final results go, one method is equivalent to the other.

Reasons for Observed Discrepancies.—Equations (*B*) and (*C*) form the basis of all present theories. They do not, and cannot, furnish a rational basis for the analysis and design of centrifugal pumps.

* See Equation No. 15, p. 12, in "Centrifugal Pumps," by Lowenstein and Crissey. Also, see Appendix II.

† This is identical with Equation No. 36, p. 62, in "Centrifugal Pumps," by Daugherty.

The reasons for this fallacy, as before, involve a misconception of the true action of a centrifugal pump. The increase in angular momentum of 1 lb. of water in passing through the impeller is,

$$\frac{1}{g}\left(C_{p2}\,V_2 - C_{p1}\,V_1\right)$$

On the assumption that the water approaches the entrance edge of the vanes in a radial direction and, hence, $C_{p1} = 0$, the work done on 1 lb. of water is found to be $\frac{1}{g}\left(C_{p2}\,V_2\right)$. This is nearly correct, but not for the reason given.

The water probably never approaches the vanes in a radial direction, but rotates with the impeller at all rates of flow. Hence, C_{p_1} is probably never zero. The error lies in the assumption that the water had some energy as it approached the runner which it received from some outside source. Naturally, if so, this should be deducted from the final energy to obtain that given by the impeller. In the centrifugal pump, however, all the energy in the water as it leaves the runner has been given by the runner and, hence, the increase in energy is $\frac{1}{g}\left(C_{p2}\,V_2\right)$.

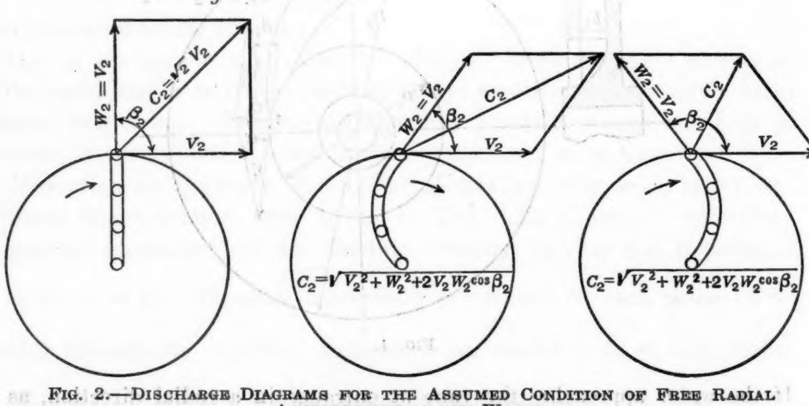


FIG. 2.—DISCHARGE DIAGRAMS FOR THE ASSUMED CONDITION OF FREE RADIAL ACCELERATION OF THE WATER.

NEW CONCEPTIONS OF PUMP ACTION

Imagine a circular disk with a radial groove from the center to the outer circumference (Fig. 2). Suppose that this disk revolves in a horizontal plane and that a very smooth round ball is introduced into the groove at the center. If there are no losses due to friction and the only forces acting are those of centrifugal force and inertia, the ball will cross the outer circumference in a radial direction relative to the moving disk with a velocity in that direction equal to its peripheral velocity, V_2*. The absolute velocity of the ball, therefore, will be the resultant of these two velocities, or $\sqrt{2} \times V_2$ in a direction inclined 45° to the tangent. The kinetic energy is clearly,

$$\frac{1}{2}\,M\,V_2^2, \text{ or } \frac{2\,V_2^2}{2\,g} = \frac{V_2^2}{g}$$

* See Appendix I for proof.

The energy given to the water, however, is only about one-half that obtained by this process. The reason is that the disk and ball and the centrifugal pump are by no means equivalent, as has been commonly supposed. For the similarity to hold it would be necessary (1) that the vanes or grooves be radial; (2) that the particles be free to accelerate radially; and (3) that the particle be introduced at the center.

Real Action of Centrifugal Pump.—None of these conditions is fulfilled in the centrifugal pump. The water contained in the impeller rotates with it and owing to centrifugal force has a tendency to move out from the center. Outside the runner is the spiral casing filled with water. The pressure in this casing acts to retard the flow from the runner. If the discharge valve is opened the pressure falls in the casing and more water flows from the runner. In the same way if the discharge valve is closed the pressure rises in the casing, thus reducing the discharge.

Thus, the flow from the runner is due to the difference between the force tending to throw the water out (centrifugal force) and the pressure in the casing tending to hold the water in the runner. This, of course, entirely prevents the free radial acceleration necessary to obtain the theoretical $\frac{V_2^2}{g}$ so often given in texts. A runner without a casing and discharging into the air might approximate this condition, but such a machine would be of no practical importance in engineering. In addition, it would be impossible to introduce each particle of water at the center of the shaft.

The result, $\frac{V_2^2}{g}$, would obviously be true only for radial vanes, at least at the discharge tips, which is only one special case. The slot in the disk might be curved forward in the direction of rotation and hence discharge the ball with an absolute velocity greater than V_2 (Fig. 2), or it might be curved backward in the opposite direction, in which case the absolute velocity is less than V_2.* If these vanes were curved forward in such a manner as to be tangent to the outer circumference the final absolute velocity would be 2 V_2 in the direction of the tangent and the energy per pound would be,

$$\frac{4\,V_2^2}{2\,g} = 2\,\frac{V_2^2}{g}$$

On the other hand, if the vanes were curved backward and tangent, the absolute velocity and, hence, the kinetic energy, would be zero. Thus, the energy put into the ball may vary according to the curvature of the vanes from zero to $2\,\frac{V_2^2}{g}$, with a mean of $\frac{V_2^2}{g}$ for radial vanes. All this presupposes that the ball is free to accelerate radially, which is never true of the water in the pump. The fact that the present pump theory is purely kinetic and depends on the previous assumption of free radial acceleration seems to have been overlooked completely. This theory would be more applicable to a pump without a casing and discharging freely into the air. Under such conditions the

* See Appendix I.

energy given to the water is entirely kinetic and can be readily obtained from the vector diagrams.

Assuming free radial acceleration, the relative velocity at discharge, regardless of the direction of the vane (forward, radial, or backward), is always equal to the peripheral speed of the vane at its outer diameter, V_2. The only factor that could make it less would be friction in passing through the impeller. In such case the kinetic energy at exit is not the sole measure of the work done on the fluid, as that done in getting it to the outer circumference should also be included. This error is commonly made in drawing the vector diagram at the discharge from the impeller.

The implied loss due to friction in the impeller passages is the hidden assumption that causes much of the difficulty in fitting theory to practice. In Fig. 3 the correct and incorrect values of the relative velocities, W_2 and W_2', and of the absolute velocities, C_2 and C_2', are shown. If the fluid is free to accelerate radially without friction loss in the impeller, the relative velocity, W_2, must equal the peripheral speed, V_2. The direction of W_2 is that fixed by the vane angle, but its magnitude is constant and equal to V_2 under the assumptions. Actually, the value of W_2 is often much less than V_2, as W_2'; but in such cases the original assumptions are not realized.

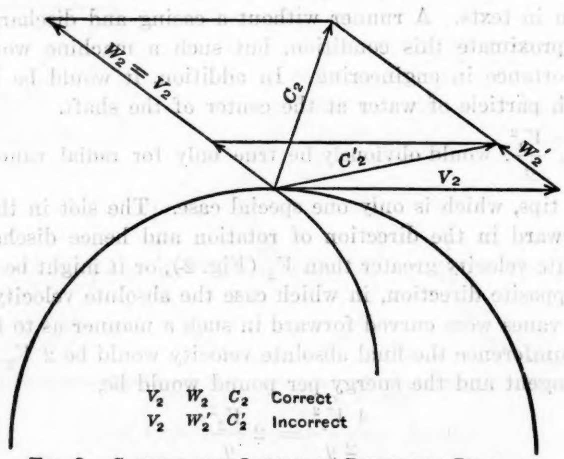


FIG. 3.—CORRECT AND INCORRECT DISCHARGE DIAGRAMS.

The restriction of the relative velocity, W_2, by the pressure in the casing is accomplished very efficiently, whereas an equal restriction caused by friction in the impeller passages would be very wasteful of energy. Similarly, the flow of water from a long pipe might be regulated very efficiently by a weir of variable height at the discharge end, but if the regulation was by a valve placed in the line, energy would be lost in friction.

EXPERIMENTAL VERIFICATION

Theoretical vs. Actual Heads.—In Fig. 4(*a*) is represented graphically the generally accepted theory of the centrifugal pump. Here the ideal quantity-head curve is shown beside the actual quantity-head curve for comparison.

Similar curves may be found* in numerous works of reference. There is an obvious inconsistency between the equations representing the head imparted to the water and their graphical representation as in Fig. 4(a). For example, the ideal quantity-head curve is seen to pass through a point representing more than double the head actually developed at the point of zero discharge—an obvious impossibility as the pressure is caused only by centrifugal force.

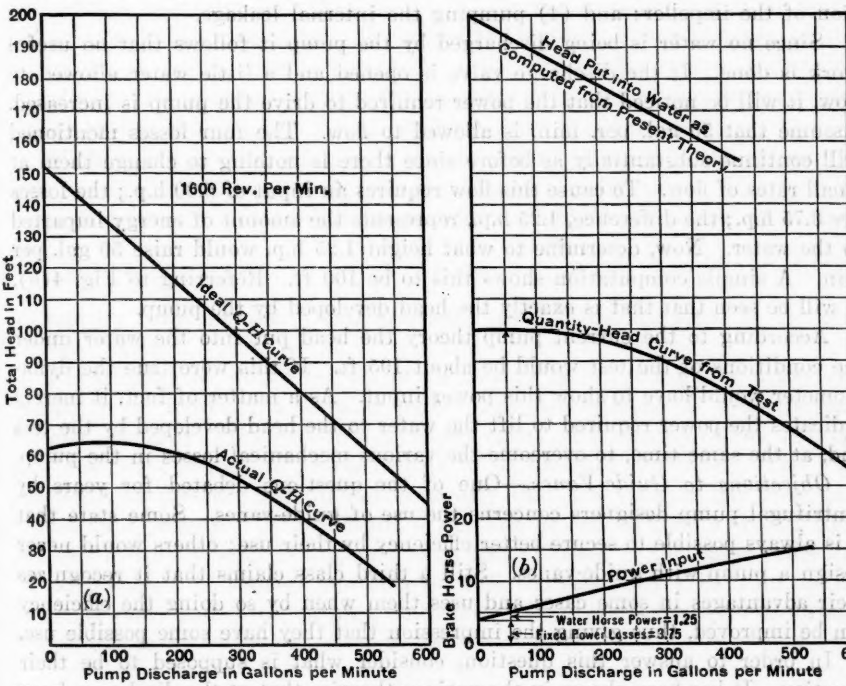


FIG. 4.—GRAPHICAL COMPARISON OF THEORETICAL AND ACTUAL HEADS IN CENTRIFUGAL PUMPS.

According to the present theory the total head is considered to be composed of two items, pressure and velocity. A measurement of the pressure indicates an average value of about $\frac{U_a^2}{2g}$ at or near the point of zero discharge. Discharge diagrams of runners show that at small rates of flow the absolute velocity, W_a, is nearly equal to U_a and, hence, the energy represented by it is nearly $\frac{U_a^2}{2g}$. This would apparently mean that the total head is approximately $\frac{U_a^2}{2g} + \frac{W_a^2}{2g}$, or very nearly $\frac{U_a^2}{g}$. As a matter of actual experience only a head of about one-half the ideal is developed by the pump. The difference is commonly said to be due to shock losses and many elaborate formulas have been developed to account for it.

* "Centrifugal Pumps," by R. L. Daugherty, pp. 76–77, Figs. 58–59.

Test Determination of Power Expenditure.—Fortunately, it is possible to settle this question beyond any doubt by a simple test in the laboratory. Referring to the characteristic curves obtained from a test of a centrifugal pump, as in Fig. 4(*b*), it will be noticed that at the point of zero discharge the power required to drive the pump is the minimum (3.75 h.p.). This power is consumed in doing the following things: (1) Overcoming the bearing friction; (2) overcoming the stuffing-box friction; (3) overcoming the disk friction of the impeller; and (4) pumping the internal leakage.

Since no water is being discharged by the pump it follows that no useful work is done. If the discharge valve is opened and a little water allowed to flow, it will be noticed that the power required to drive the pump is increased. Assume that 50 gal. per. min. is allowed to flow. The four losses mentioned will continue substantially as before since there is nothing to change them at small rates of flow. To cause this flow requires an input of 5.00 h.p.; the losses are 3.75 h.p.; the difference, 1.25 h.p., represents the amount of energy imparted to the water. Now, determine to what height 1.25 h.p. would raise 50 gal. per min. A simple computation shows this to be 100 ft. Referring to Fig. 4(*b*), it will be seen that that is exactly the head developed by the pump.

According to the current pump theory the head put into the water under the conditions of the test would be about 195 ft. If this were true the dynamometer would have to show this power input. As a matter of fact, it merely indicates the power required to lift the water to the head developed by the test and, at the same time, to overcome the various mechanical losses in the pump.

Objections to Guide-Vanes.—One of the questions debated for years by centrifugal pump designers concerns the use of guide-vanes. Some state that it is always possible to secure better efficiency by their use; others would never design a pump with guide-vanes. Still a third class claims that it recognizes their advantages in some cases and uses them when by so doing the efficiency can be improved, thus giving the impression that they have some possible use.

In order to answer this question, consider what is supposed to be their function. It has been shown in the various theories that at the discharge from the runner the water has a pressure head, $\frac{P_2}{\gamma}$, as well as a velocity head, $\frac{C_2^2}{2\,g}$. It is often stated that this velocity head, $\frac{C_2^2}{2\,g}$, will be lost in shock, friction, etc., unless suitable provision is made for its recovery. The commonly accepted idea has been that at very small rates of flow this energy is about equally divided between pressure and velocity and that the entire velocity head under these conditions was apt to be lost unless special provision was made in the form of guide-vanes or diffusor vanes. Such vanes were designed so as to receive the absolute velocity of the water. By increasing the area of the water passages between them, the velocity was expected to be converted most efficiently into pressure.

Evidently, from the previous discussion, the use and benefits of guide-vanes are founded on a misconception of the theory. In a centrifgal pump it is absolutely wrong to assume that the total energy is represented by the pressure in the casing plus the kinetic energy of the absolute velocity of discharge

As the water is confined to a circular path, its kinetic energy is necessarily converted into pressure without any assistance from the guide-vanes. In fact, they often serve to obstruct the natural flow of water, which entirely defeats their purpose and renders them useless. From the point of view of practice equal or better efficiency can be obtained with a properly designed volute.

Purpose of Casing.—What has just been said with regard to the use of guide-vanes may also be extended to include the so-called volute or spiral casing into which the runner discharges the water. The advantage of the volute casing is supposed to lie chiefly in permitting the efficient conversion of velocity into pressure through the recovery of a part of the energy represented by the absolute velocity of discharge from the runner.

While it is true that velocity is converted into pressure in the casing, yet it is not at all in the manner or for the reasons usually given. If a body of water is set in rotation so that the peripheral speed of the outermost particle is V, then a pressure will be developed equal to $\frac{V^2}{2g}$, a case of perfect conversion of velocity into pressure. This is exactly what is taking place in the centrifugal pump.

Apparently, the only function of the casing is to restrain the water and compel it to move in a circular path. The development of pressure then follows naturally. The pressure obviously exists only because of the velocity, and one may be said to be equivalent to or measure the other. The very existence of pressure here indicates clearly that the velocity has been converted into pressure. It is difficult to see how it can be done a second time, yet this is exactly what is supposed to be accomplished by the guide-vanes and spiral casings. This shows how hopeless it is to expect any gain from the use of guide-vanes and spiral casings, since their advantage is based on conditions which can easily be proved never to have existed.

New Analysis of the Centrifugal Pump

The Impeller and Its Action.—A pump is fundamentally a device for increasing the energy of the fluid passing through it. In the centrifugal pump this energy is first in the kinetic, and, later, in the pressure, form. The conversion of velocity into pressure takes place in the casing as will be shown. For the present, however, consider the action of the runner quite apart from that of the casing.

Imagine a runner arranged so as to receive the water at its suction entrance and discharge it into the air—a device resembling a lawn sprinkler.

A particle of water, initially at rest at the level of the source of supply, soon leaves the runner with high velocity and, therefore, possesses kinetic energy. All this energy must necessarily have come from the impeller. Numerically, it is equal to $\frac{C_2^2}{2g}$ per lb. of water, neglecting any slight static head which might be present in addition to the velocity head, $\frac{C_2^2}{2g}$. The

design of the vanes at the discharge from the runner determines the amount of energy given to the water. This can be seen by reference to Fig. 5. If the vanes are curved forward in the direction of rotation the kinetic energy given to the water will increase as the discharge from the pump is increased. On the other hand for backward curved vanes the reverse will be true.

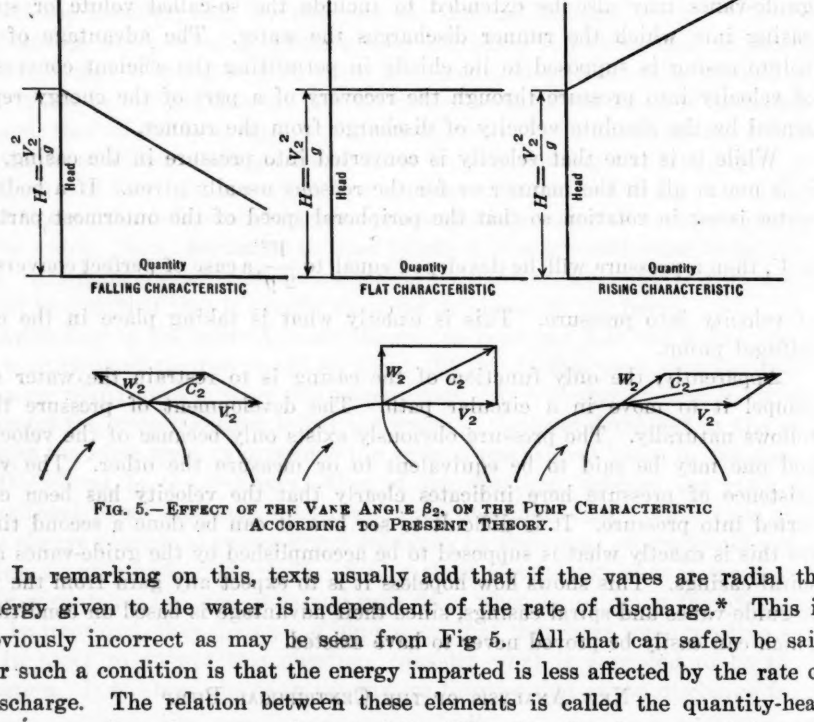


FIG. 5.—EFFECT OF THE VANE ANGLE β_2, ON THE PUMP CHARACTERISTIC ACCORDING TO PRESENT THEORY.

In remarking on this, texts usually add that if the vanes are radial the energy given to the water is independent of the rate of discharge.* This is obviously incorrect as may be seen from Fig 5. All that can safely be said for such a condition is that the energy imparted is less affected by the rate of discharge. The relation between these elements is called the quantity-head curve, or often the pump characteristic (Fig. 5). In the case of forward curved vanes the head increases as the rate of discharge, and the runner is said to have a rising characteristic. If the vanes are radial it is usually (incorrectly) assumed that the head is independent of the discharge, and such runners are said to have a flat characteristic. When the vanes are curved backward the head decreases as the rate of flow, and the characteristic is said to be steep or falling.† All this, with some exceptions, would be substantially correct for a pump without a casing, but text-books fail to state that qualification.

As a matter of fact in centrifugal pumps as built commercially the vane angle is only one of several factors that determine the pump characteristic. It does not, as many imagine, entirely control the shape of the quantity-head curve. If there were no casing and the water were free to discharge into the

* See "Centrifugal Pumping Machinery," by DeLaval, p. 58, Fig. 56.
† "Centrifugal Pumps," by R. L. Daugherty, p. 63, Fig. 55.

air the total energy imparted to the water could be measured by the work done on it in giving it its absolute kinetic energy, $\frac{C_2^2}{2g}$ per lb. of water.

Effects of Casing.—As a matter of fact pumps always have a casing, generally spiral in design, which completely encloses the impeller. The water in this casing between the runner and the side-walls and also in the volute is set in rotation by the impeller. The water discharged from the runner with an absolute velocity, C_2, has a relatively large tangential component. Mingling with that circling in the casing, it assists in maintaining the revolution. As the water is confined to a somewhat circular path a pressure is created in the casing due to centrifugal force.

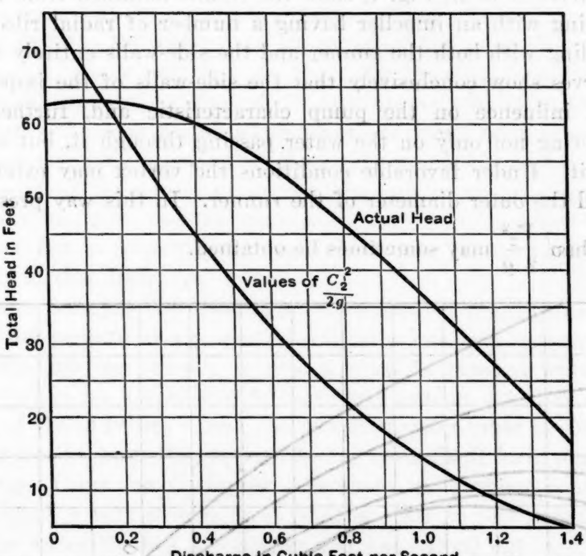


FIG. 6.—COMPARISON BETWEEN ACTUAL HEAD DEVELOPED AND THAT COMPUTED FROM VALUES OF THE ABSOLUTE VELOCITY OF DISCHARGE FROM THE RUNNER, OR C_2.

To use a very homely illustration this might be compared to a paddle stirring the water in a cylindrical tank assisted by a jet of water applied tangentially. The final velocity of rotation is the result of these two causes; but it is difficult to separate the effect of each. Basing the theoretical power input on the value of the absolute velocity of discharge from the runner (C_2) much less power is obtained than is actually supplied as determined by an accurate test. In Fig. 6 the value of $\frac{C_2^2}{2g}$ has been plotted as a function of the discharge. The actual quantity-head curve is also shown for comparison. Actually, much more energy has been imparted to the water than can be accounted for on the basis of the absolute velocity head, $\frac{C_2^2}{2g}$. It seems more

nearly to approach the constant value, $\frac{V_2^2}{2g}$, but probably is actually some intermediate value yet to be determined.

The runner acts not only on the water passing through it, but also on that outside, between it and the side-walls of the casing. The water in this space is set in rotation by the dragging action of the side-walls of the impeller. As a result, a vortex is formed within the case quite independent of whether or not the pump is actually discharging any water.

The nature of the side-walls of the impeller and case determines the extent to which this vortex may be formed. A number of experiments were performed to determine the effect of various conditions of the side-walls of the impeller. Curves *A* to *E*, Fig. 7, show the results obtained from a succession of tests starting with an impeller having a number of radial ribs cast on its sides and ending with both the runner and the side-walls entirely smooth.

These curves show conclusively that the side-walls of the impeller have a very marked influence on the pump characteristic and, further, that the impeller is acting not only on the water passing through it, but also on that surrounding it. Under favorable conditions the vortex may extend into the casing beyond the outer diameter of the runner. In this way pressures somewhat higher than $\frac{V_2^2}{2g}$ may sometimes be obtained.

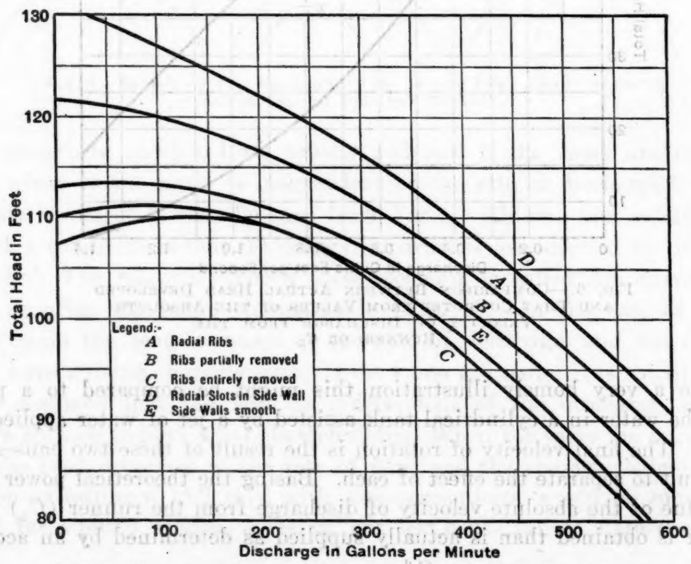


FIG. 7.—CHARACTERISTIC CURVES OF A 4-INCH CENTRIFUGAL PUMP AT 1750 REVOLUTIONS PER MINUTE, SHOWING VARIATION IN THE CHARACTERISTIC CAUSED BY DIFFERENT SIDE WALLS OF IMPELLER.

The absolute velocity of the water as it leaves the impeller will assist in maintaining this rotation and the vortex results from their combined action. The outer diameter of this vortex, which determines the pressure developed by the pump, will be called D_3 and the peripheral speed of the water at that

point, V_3. It is likely that D_3 and, hence, V_3 will vary somewhat with the pump discharge. Owing to the fact that the water is confined to a circular path this velocity, V_3, is converted into pressure and causes the water to rise to a height, $\frac{V_2^2}{2\,g}$.

Action After Flow Begins.—Consider a pump running at a certain speed, but with its discharge valve closed. The water in the impeller and the casing is set in rotation, creating a pressure in the pump, but flow from the runner is prevented owing to the pressure of the water in the casing. Now, assume that the discharge valve is opened a trifle to permit a slight amount of flow. The pressure in the casing falls, and the pressure opposing the flow from the impeller decreases, but the centrifugal force tending to throw the water out of the impeller remains the same since the speed of the pump is constant.

Consequently, water will flow in from the impeller into the casing, the quantity depending on the difference in pressure as well as on the area of the water passages. As soon as the water begins to move relative to the impeller friction comes into play opposing the flow. The pump will adjust itself to these conditions so that the resultant of the pressure in the casing, the drop in pressure due to the flow, and the frictional losses in the impeller will equal the total pressure due to centrifugal force. From now on the pump will operate continuously at this discharge.

On further opening the discharge valve similar re-adjustments continue until the entire pressure due to centrifugal force has been consumed in forcing the water through the runner, or else a point is reached where the atmospheric pressure is no longer able to force any greater quantity of water into the suction of the impeller. Thus, the flow from the runner is simply governed by the difference between the pressure due to centrifugal force and the pressure in the casing. Once this is clearly understood the problem is merely that of the flow through a very special form of orifice due to this pressure difference.

After the water has left the runner and has passed out into the casing, it must flow through the so-called throat of the pump before passing the discharge flange. This throat is merely the point of maximum cross-section of the volute. All the water discharged from the runner passes through this, a third orifice.

Pump Considered as Three Orifices.—Hence, the entire problem of the centrifugal pump is that of flow through three orifices in series. The first is at the suction entrance to the impeller; and this determines the maximum capacity of the pump. The second is at the discharge from the runner, and the third is at the throat of the pump. Each of these orifices or points of restriction has its own characteristics and their combined effect determines the pump characteristic.

Consider that the pump is running at a constant speed. The pressure developed by centrifugal force, tending to force the water out of the impeller, is constant and is quite independent of the flow or the amount of it. Centrifugal force is equal to $\frac{M\,V^2}{r}$; evidently, neither V nor r is greatly changed

by the flow of water and the mass, M, need not be considered in the determining the energy input per pound of water. In other words, if there were no loss of pressure in the impeller and casing, the pump would deliver any quantity of water within the limit of the suction capacity at a constant pressure of approximately $\frac{V_2^2}{2g}$. A drop in pressure at the discharge flange is necessary if any water is to flow from the impeller.

The quantity that will flow depends on the area of the water passages; and its velocity, on the pressure difference established. The velocity is, therefore, quite independent of the quantity being discharged. The quantity is the product of the velocity and the area of the water passages measured perpendicularly to it.

The amount the pressure at any point falls below the horizontal line tangent to the quantity-head curve at "shut-off" is very nearly that required to force this quantity of water through the pump. The total drop in pressure is (1) that required to cause the given quantity of water to flow; and (2) that required to overcome the hydraulic losses. These are fundamentally different in their nature and yet produce similar effects on the pump characteristic.

Analogy Between Pump and Electric Generator.—The drop in pressure required to produce flow is an example of a loss in pressure without any loss in power. A similar instance occurs in the field of electricity. An electric generator would deliver a constant voltage at its terminals were it not for two principal losses. These are the drops in pressure due to armature reaction and to internal resistance. The drop in pressure due to armature reaction causes no loss of energy, whereas that due to resistance wastes energy in the form of heat. A loss of pressure that does not involve a loss of power is called a pressure drop, while a loss of power is called a pressure loss. These same terms might apply to the pump theory. In fact, many close analogies exist between the centrifugal pump and a separately excited shunt generator.

In the pump the drop in pressure without loss of power merely means that a greater pressure drop is required to discharge a given quantity of water through a small orifice than for one of larger area; or that for a given pressure drop, more water will flow through a large orifice than a small one. This does not mean that the large orifice is necessarily more efficient than the smaller one for both may have a coefficient of nearly 100 per cent.

When water flows through the runner it has to overcome the frictional losses. These produce a drop in pressure, which is also a loss of power. Since the length of the path of the water in the impeller is short and the water ways are smooth, this loss of power due to friction usually is not large.

Application of Theory to Determine Pump Characteristic

To make the new theory clear and indicate its application to practice, three characteristic curves, showing the relation between the capacity and

head, at constant speed, have been worked out for different typical pumps. These machines are of usual design and construction and of well-known makes. Fig. 8, showing typical views, serves to illustrate the three points of control or orifices. The first is at the entrance to the impeller-vane passages, and its area is equal to the product of the dimensions, B and D, and the total number of vane passages. The second is at the discharge from the impeller-vane passages; its area is the product of the dimensions, A and C, and the total number of vane passages. The third is at the throat of the pump; its area is usually that of a circle of diameter, E (Fig. 8).

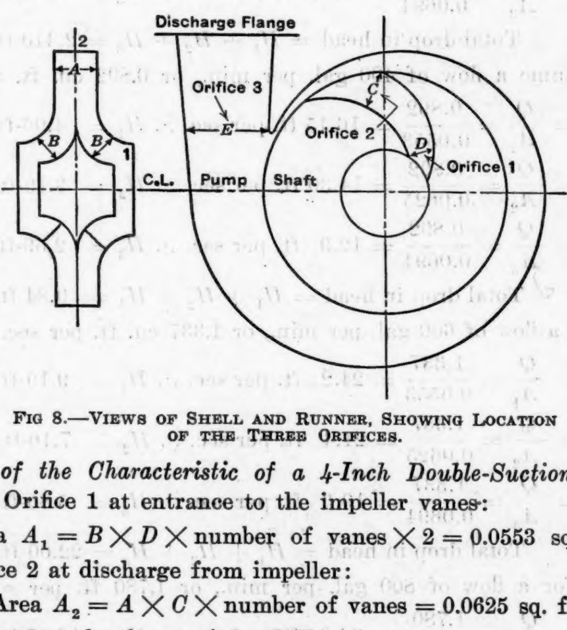


FIG 8.—VIEWS OF SHELL AND RUNNER, SHOWING LOCATION OF THE THREE ORIFICES.

Analysis of the Characteristic of a 4-Inch Double-Suction Centrifugal Pump.—For Orifice 1 at entrance to the impeller vanes:

$$\text{Area } A_1 = B \times D \times \text{number of vanes} \times 2 = 0.0553 \text{ sq. ft.}$$

For Orifice 2 at discharge from impeller:

$$\text{Area } A_2 = A \times C \times \text{number of vanes} = 0.0625 \text{ sq. ft.}$$

For Orifice 3 at the throat of the shell:

$$\text{Area } A_3 = \frac{\pi E^2}{4} = 0.0694 \text{ sq. ft.}$$

The velocity through the orifice is equal to $\frac{Q}{A}$, hence the velocity through Orifice 1 is,

$$\frac{Q}{A} = \frac{Q}{0.0553} = C_1 \text{ ft. per sec.}$$

the velocity through Orifice 2 is,

$$\frac{Q}{A_2} = \frac{Q}{0.0625} = C_2 \text{ ft. per sec.}$$

and the velocity through Orifice 3 is,

$$\frac{Q}{A_3} = \frac{Q}{0.0694} = C_3 \text{ ft. per sec.}$$

The drop in head or pressure required to produce the speed, C_1, C_2, or C_3, can be computed from the well-known formula, $H = \frac{C^2}{2\ g}$.

If a flow of 200 gal. per min. (= 0.446 cu. ft. per sec.) be assumed:

$$C_1 = \frac{Q}{A_1} = \frac{0.446}{0.0553} = 8.05 \text{ ft. per sec.} \therefore H_1 = 1.010\text{-ft. drop}$$

$$C_2 = \frac{Q}{A_2} = \frac{0.446}{0.0629} = 7.13 \text{ ft. per sec.} \therefore H_2 = 0.765\text{-ft. drop}$$

$$C_3 = \frac{Q}{A_3} = \frac{0.446}{0.0694} = 6.43 \text{ ft. per sec.} \therefore H_3 = 0.645\text{-ft. drop}$$

$$\text{Total drop in head} = H_1 + H_2 + H_3 = 2.410 \text{ ft.}$$

Next, assume a flow of 400 gal. per min., or 0.892 cu. ft. per sec.:

$$C_1 = \frac{Q}{A_1} = \frac{0.892}{0.0553} = 16.15 \text{ ft. per sec.} \therefore H_1 = 4.06\text{-ft. drop}$$

$$C_2 = \frac{Q}{A_2} = \frac{0.892}{0.0625} = 14.30 \text{ ft. per sec.} \therefore H_2 = 3.19\text{-ft. drop}$$

$$C_3 = \frac{Q}{A_3} = \frac{0.892}{0.0694} = 12.9 \text{ ft. per sec.} \therefore H_3 = 2.59\text{-ft. drop}$$

$$\text{Total drop in head} = H_1 + H_2 + H_3 = 9.84 \text{ ft.}$$

Next, for a flow of 600 gal. per min., or 1.337 cu. ft. per sec.:

$$C_1 = \frac{Q}{A_1} = \frac{1.337}{0.0553} = 24.2 \text{ ft. per sec.} \therefore H_1 = 9.10\text{-ft. drop}$$

$$C_2 = \frac{Q}{A_2} = \frac{1.337}{0.0625} = 21.4 \text{ ft. per sec.} \therefore H_2 = 7.10\text{-ft. drop}$$

$$C_3 = \frac{Q}{A_3} = \frac{1.337}{0.0694} = 19.3 \text{ ft. per sec.} \therefore H_3 = 5.80\text{-ft. drop}$$

$$\text{Total drop in head} = H_1 + H_2 + H_3 = 22.00 \text{ ft.}$$

Finally, for a flow of 800 gal. per min., or 1.780 ft. per sec:

$$C_1 = \frac{Q}{A_1} = \frac{1.780}{0.0553} = 32.2 \text{ ft. per sec.} \therefore H_1 = 16.10\text{-ft. drop}$$

$$C_2 = \frac{Q}{A_2} = \frac{1.780}{0.0625} = 28.5 \text{ ft. per sec.} \therefore H_2 = 12.60\text{-ft. drop}$$

$$C_3 = \frac{Q}{A_3} = \frac{1.780}{0.0694} = 25.7 \text{ ft. per sec.} \therefore H_3 = 10.30\text{-ft. drop}$$

$$\text{Total drop in head} = H_1 + H_2 + H_3 = 39.00 \text{ ft.}$$

The head developed by this pump at "shut off", or the point of no delivery, is 137 ft. as found by a test.

For various rates of flow the head is obtained by deducting the total drop, as calculated for any assumed rate of flow, from the shut-off head of 137 ft. Hence,

The head at 200 gal. per min. = 137.0 — 2.41 = 134.6 ft.
" " " 400 gal. per min. = 137^2 — 9.84 = 127.2 ft.
" " " 600 gal. per min. = 137^2 — 22.00 = 115.0 ft.
" " " 800 gal. per min. = 137 — 39.00 = 98.0 ft.

This is shown graphically in Fig. 9 by the full line. The actual quantity-head curve, as obtained from the test (dash line), is shown for comparison. Evidently, the two are in substantial agreement. The slight differences may be due to the fact that the dimensions used in the calculations were obtained from the design drawings and not from actual pump measurements.

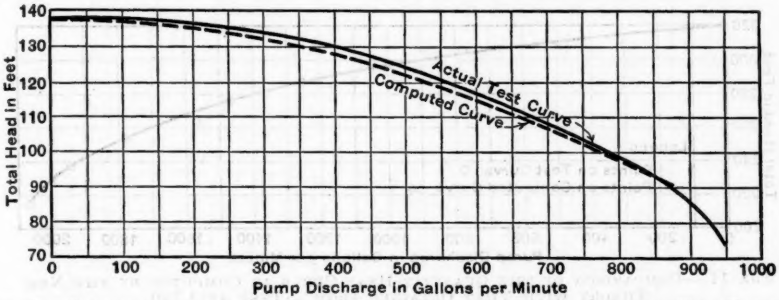


FIG. 9.—COMPARISON OF THE QUANTITY-HEAD CURVE AS COMPUTED BY THE NEW THEORY WITH THAT OBTAINED FROM A TEST AT 1 750 REVOLUTIONS PER MINUTE.

Similarly, Fig. 10 shows the analysis for a 3-in. pump for low or moderate heads and Fig. 11, that for a 6-in. high-head pump. The method is seen to yield satisfactory agreement for all three types.

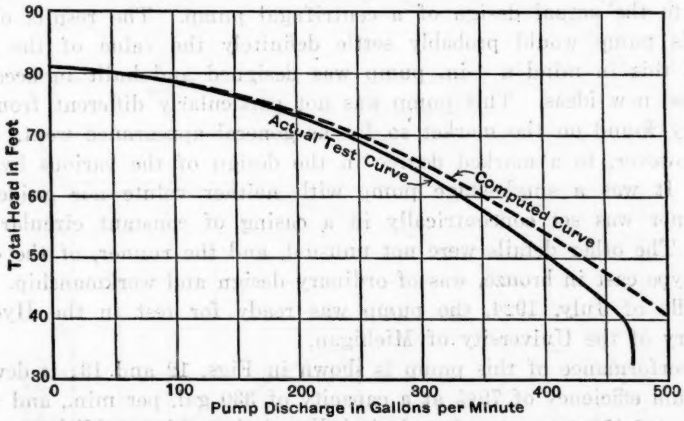


FIG. 10.—COMPARISON OF THE QUANTITY-HEAD CURVE AS COMPUTED BY THE NEW THEORY WITH THAT OBTAINED FROM A TEST AT 1 750 REVOLUTIONS PER MINUTE.

This should make clear the complete action of the centrifugal pump. It is then easy to explain the general effect of various changes in the design of a pump and to calculate the approximate numerical or quantitative effect of such changes on the pump characteristic. For obvious reasons this was not always possible with the previous theory.

APPLICATION OF NEW THEORY TO DESIGN

It would be a logical question whether this new theory is likely to result in any radical changes in the design or construction of centrifugal pumps.

The fallacy on which the use of both the guide-vanes, or diffusors, and spiral casings, or volutes, is based, has already been mentioned. The new

theory indicates that all these accessories are not only useless but possibly even wrong. If this is correct it should be possible to obtain just as good results with a runner mounted concentrically in a plain circular casing of constant cross-section as with the most elaborately calculated spiral casings, volutes, diffusors, etc.

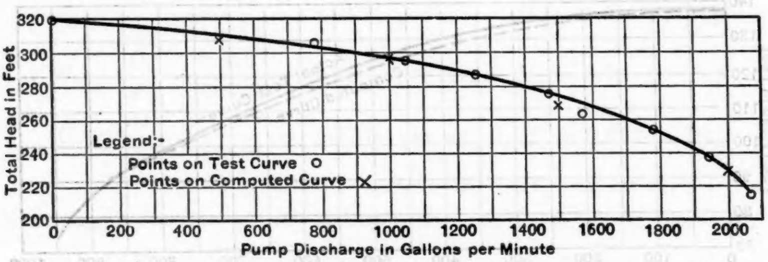


FIG. 11.—COMPARISON OF THE QUANTITY-HEAD CURVE AS COMPUTED BY THE NEW THEORY WITH THAT OBTAINED FROM A TEST AT 1 750 REVOLUTIONS PER MINUTE.

Results of First Design.—As this conclusion was a very radical departure from accepted current views it was felt that the final proof needed was to apply these principles (and others developed in connection with the new theory) to the actual design of a centrifugal pump. The results obtained with this pump would probably settle definitely the value of the theory.

With this in mind a 4-in. pump was designed and built in accordance with these new ideas. This pump was not particularly different from those ordinarily found on the market so far as general appearance went. It did differ, however, to a marked degree in the design of the various hydraulic details. It was a single-stage pump with neither volute nor guide-vanes. The runner was set concentrically in a casing of constant circular cross-section. The other details were not unusual, and the runner, of the double-suction type cast in bronze, was of ordinary design and workmanship. About the middle of July, 1924, the pump was ready for test in the Hydraulic Laboratory of the University of Michigan.

The performance of this pump is shown in Figs. 12 and 13; it developed a maximum efficiency of 79% at a capacity of 330 gal. per min., and a head of 67 ft. at 1 450 rev. per min. It is believed that this establishes a record for both American and European practice for a pump of such capacity. The usual efficiency obtained under such conditions is only about 60 per cent.

The extreme flatness of the efficiency curve in Fig. 12 should be noted, also the broadness of the equal efficiency curves in Fig. 13. This indicates that it has been possible to develop extremely high efficiencies without any sacrifice of other desirable features.

Further Verification.—Those engaged in the design, manufacture, and sale of centrifugal pumps are likely to demand proof of an experimental nature rather than mathematical deductions no matter how skillfully and carefully they may be drawn. The writer, therefore, has made every effort to include actual test results in conjunction with his previous more or less mathematical conclusions.

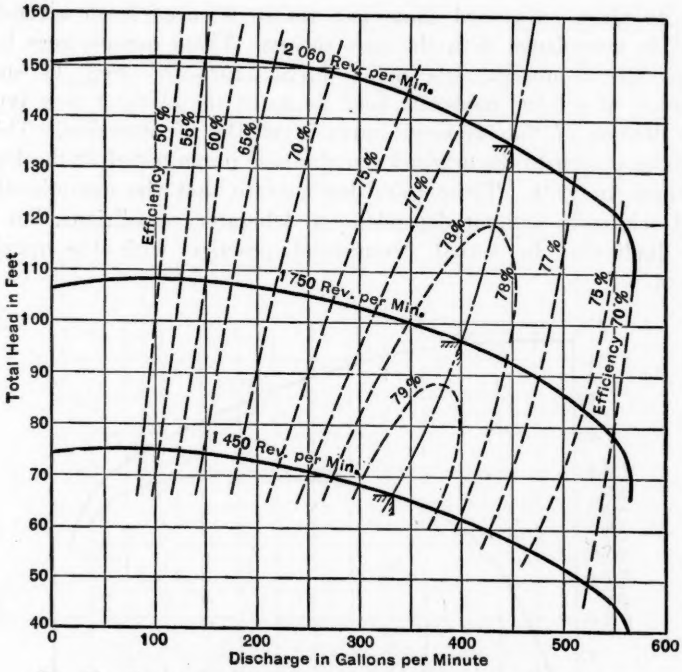


FIG. 12.—CHARACTERISTIC CURVE OF A 4-INCH CENTRIFUGAL PUMP.

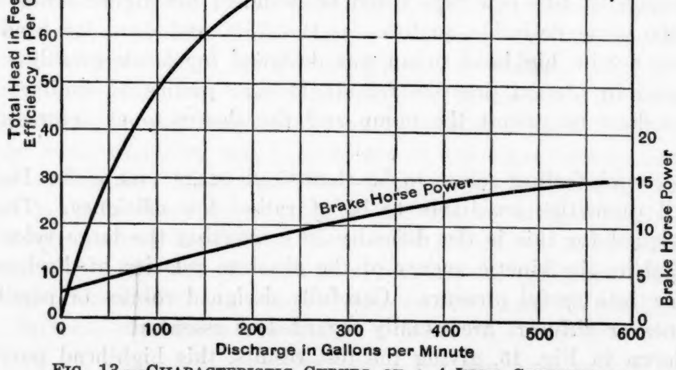


FIG. 13.—CHARACTERISTIC CURVES OF A 4-INCH CENTRIFUGAL PUMP AT 1 750 REVOLUTIONS PER MINUTE.

Through the courtesy of one of the large pump manufacturers, it is now possible to place on record some test results secured from several pumps designed in accordance with the new theory. These pumps were built and tested at the manufacturer's plant during 1925-27. Fig. 14 shows the performance of a 3-in., moderate head pump of the circular case type. The extreme flatness of the efficiency curve is worthy of attention. This pump is similar in design to the original circular-case pump tested at the University of Michigan in 1924. These tests demonstrate that the exceptional results obtained originally are not dependent on laboratory conditions, but may be readily duplicated in actual commercial practice with the regular test equipment.

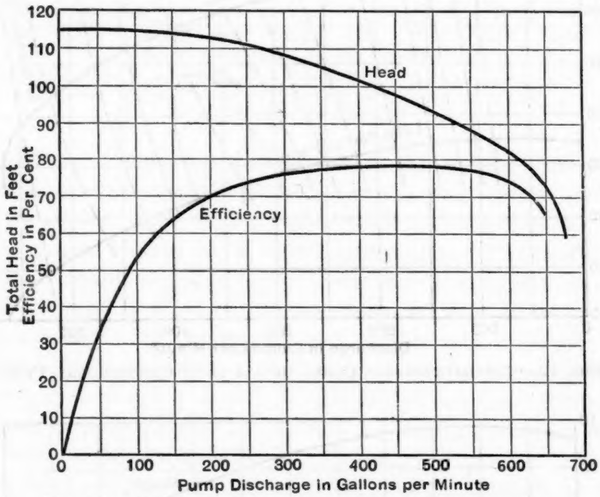


FIG. 14.—TEST CURVES SHOWING THE PERFORMANCE OF A 3-INCH CIRCULAR CASE PUMP AT 1 750 REVOLUTIONS PER MINUTE.

Results with High-Head Pump.—Some question next arose as to whether or not pumps of this new type could be designed for higher heads and still exhibit the same desirable qualities as the 3-in. and 4-in. low-head pumps. Therefore, a 3-in. highhead pump was designed for heads greatly exceeding those found in present practice for single-stage pumps of similar capacity. This was done to submit the pump and the theory to as severe a test as possible.

The general feeling seems to be that single-stage pumps for high heads and small capacities are likely to be of rather low efficiency. The reason often assigned for this is the difficulty in converting the large velocity head represented by the kinetic energy of the absolute velocity of discharge from the runner into useful pressure. Carefully designed volutes or possibly even guide-vanes or diffusors are usually regarded as essential.

As shown in Fig. 15, giving the test results, this high-head pump developed its maximum efficiency (72%) at a capacity of about 550 gal. per min. against a total head of 260 ft., at 1 750 rev. per min. Such high efficiency

is remarkable considering the fact that other pumps used for heads as much as 100 ft. less show efficiencies not greatly exceeding 60 per cent.

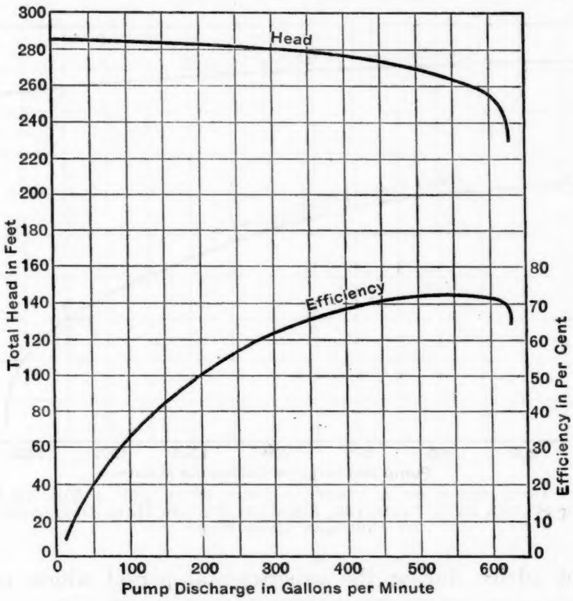


FIG. 15.—CURVES SHOWING PERFORMANCE OF A 3-INCH CIRCULAR CASE HIGH-HEAD PUMP AT 1 750 REVOLUTIONS PER MINUTE.

From this it would appear that the theory and design hold equally well for both high and low heads. It is interesting to note that the pump based on the new theory has neither volute nor guide-vanes and is designed, according to present standards, in about the worst possible way. Actually, it is doing the work for which pumps of from two to four stages are often required and even then it greatly exceeds them in efficiency.

Performance Compared with Other Designs.—In spite of the probable general belief to the contrary the writer is of the opinion that a pump of the circular-case type permits much greater efficiency in the conversion of velocity into pressure than the volute type. As evidence of this, Fig. 16 shows the performance of a circular-case pump designed by the writer compared with that of a standard volute type for similar capacity. Both runners are of the same diameter and both rotate at the same speed; that is, the peripheral speeds are the same. The circular-case pump at a capacity of 1 300 gal. per min. develops a head of 331 ft., as compared with 220 ft. for the volute pump. This is an increase of 111 ft., or about 50 per cent. The capacity of 1 300 gal. per min. is about that of maximum efficiency for each pump. Comparison of other circular-case pumps showed somewhat the same increase over the volute type.

It is hoped that engineers will freely contribute comments on these ideas, favorable or otherwise. Constructive criticism is particularly welcome.

The writer wishes to acknowledge his indebtedness to his many colleagues at the University of Michigan and elsewhere who gave so generously of

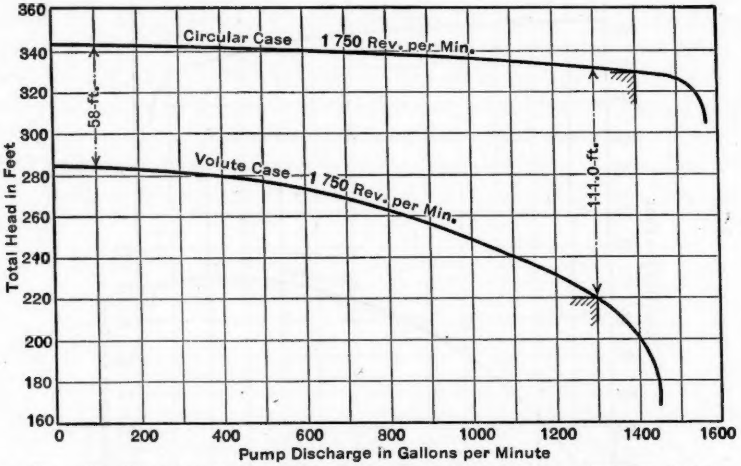


FIG. 16.—COMPARISON OF A CIRCULAR CASE PUMP AND A VOLUTE CASE PUMP OF THE SAME DIAMETER, SHOWING EXCESS HEAD DEVELOPED BY CIRCULAR CASE PUMP.

their time and advice during the experimental period which preceded the writing of this paper.

APPENDIX I*

ACCELERATION CAUSED BY ROTATION

Given a rotating disk of radius, r, turning at an angular velocity, ω. A particle is constrained to slide along a groove from the center, O, to a point, C, on the outer circumference. It is required to determine the relative velocity of the particle with respect to the disk, assuming that the particle starts from rest at the center, O.

From the general theory,† if,

M = the mass of the particle.

j_r = the relative acceleration.

F_b = the body force (in this case, the instantaneous value of the centrifugal force at any point in the path, OC).

F_c = the complementary force taken up by the constraint (in this case, $F_c = O$, since the force is normal to the path and hence has no effect on the motion of the particle).

F = the applied force (in this case there is none).

V_r = the relative velocity of the particle to the disk,

* This derivation was furnished through the courtesy of Professor Peter Field, of the College of Engineering, University of Michigan.

† "Introduction to Analytic Mechanics," by Ziwet and Field, p. 355.

Then,

$$Mj_r = F + F_b + F_c$$

or,

$$\frac{M\, d\, V_r}{d\, t} = F + F_b + F_c$$

Multiply by V_r; then,

$$M V_r \frac{d\, V_r}{d\, t} = (F + F_b + F_c)\, V_r$$

Multiply by $d\, t$; then

$$M\, V_r\, d\, V_r = (F + F_b + F_c)\, d_r$$

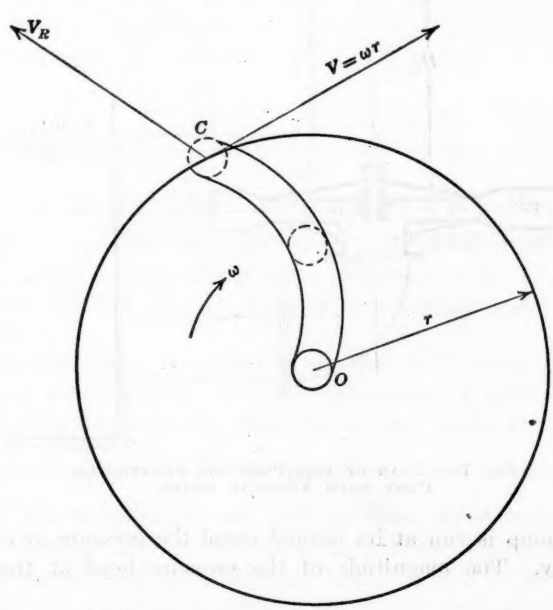


FIG. 17.—ACCELERATION CAUSED BY ROTATION.

But F and F_c are zero in this case; hence,

$$M\, V_r\, d\, V_r = \omega^2\, r^2\, d_r$$

This is the differential of,

$$\frac{1}{2}\, M\, V_r^{\,2} = \frac{1}{2}\, M\, \omega^2\, r^2$$

Therefore,

$$V_r^{\,2} = \omega^2\, r^2;\ \text{or,}\ V_r = \omega\, r$$

In other words, if the particle starts from rest at O, it will accelerate to a relative velocity, $V_r = \omega\, r$, at the outer circumference, regardless of the curvature of the path.

APPENDIX II

Derivation of the Fundamental Equations*

Fig. 18 shows schematically an arrangement of a low-pressure centrifugal pump having a vertical shaft. In every part of the pump proper there exists a pressure greater than atmospheric, due to its depth below the level of supply. Imagine small pipes connected to the inside of the pump at points s, e, a, l, and d. As all these points lie in a horizontal plane, there exists at all points the same pressure head, h_a, when the pump is standing still.

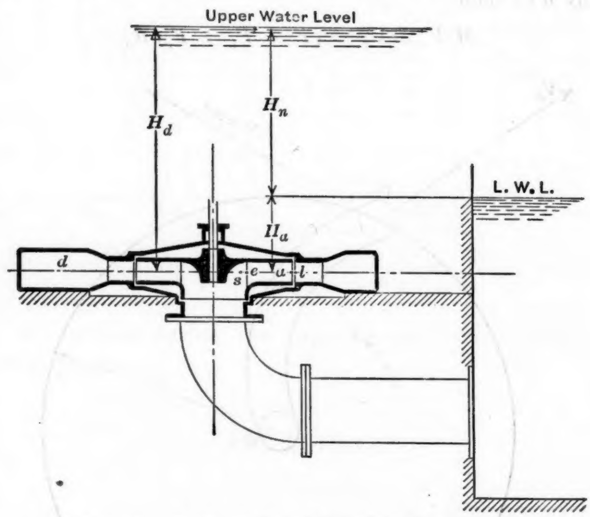


Fig. 18.—Plan of Low-Pressure Centrifugal Pump with Vertical Shaft.

When the pump is run at its normal speed the pressure at each of these points will vary. The magnitude of the pressure head at these points is to be found.

The following derivation of the so-called "condition equation" is an old method used for determining the fundamental equation of water turbines. It gives a clear conception of the pressure and velocity distribution in the interior of a centrifugal pump. The pump shown in Fig. 18 is placed below the level of the supply water in order that all the pressure heads may be positive, that is, so that no suction exists. Of course the equations can be derived also for a pump placed above the level of the water.

When the pump is running there will be a pressure head, h_s, in the pipe at Point s. When the pump is at rest, it is less by an amount equal to a velocity head, $\frac{w_s^2}{2\,g}$, plus a friction head, $p_s\,H_n$. The friction head is dissipated in forcing the water through the entrance pipe up to Point s. The

* Reproduced substantially from "Centrifugal Pumps," by Lowenstein and Crissey.

velocity in the entrance pipe, or suction tube, is designated by w_s, and the velocity head, $\frac{w_s^2}{2g}$, represents the loss of pressure head necessary to obtain the velocity, w_s, in the pipe.

Let H_n be the effective pressure or desired head of the pump. The condition equation may now be expressed by,

$$h_w = h_s + \frac{w_s^2}{2g} + p_s H_n \dots\dots\dots\dots\dots\dots (1)$$

The next point, e, is taken directly at the entrance to the impeller. At this point a pressure head, h_e, and a velocity head, $\frac{w_e^2}{2g}$, exist, w_e being the absolute velocity of the water at this point. In the water flowing from Point s to Point e shock losses occur at the entrance to the channels of the impeller, which can be expressed as a friction head, $p_e H_n$, hence,

$$h_s + \frac{w_s^2}{2g} = h_e + \frac{w_e^2}{2g} + p_e H_n \dots\dots\dots\dots\dots\dots (2)$$

If, at Point e, the relative velocity, v_e, is considered instead of the absolute velocity, w_e, then a relative velocity head, $\frac{v_e^2}{2g}$, is obtained, while the pressure head, h_e, remains the same. It is understood, of course, that absolute velocity is the velocity of the water relative to some fixed or stationary point, and that relative velocity is the velocity of the water relative to the rotating parts.

The water now passes through the impeller to Point a, which is directly at the exit of the impeller. Through the action of centrifugal force the water, in flowing through the channels of the impeller, has its pressure increased by the value, $\frac{u_a^2 - u_e^2}{2g}$, the quantities, u_a and u_e, being the peripheral velocities of the impeller at exit and entrance, respectively. Due to the friction of the water against the impeller walls and of the individual water particles among themselves, a friction head, $p_a H_n$, is lost. At Point a, a pressure head, h_a, and a velocity head, $\frac{v_a^2}{2g}$, exist, v_a being the relative exit velocity. Therefore, Equation (3) may be written:

$$\frac{v_e^2}{2g} + \frac{u_a^2 - u_e^2}{2g} = h_a + \frac{v_a^2}{2g} + p_a H_n \dots\dots\dots\dots\dots\dots (3)$$

At Point l, which is directly at entrance to the guide-vanes, there exists a pressure head, h_l, and also a velocity head, $\frac{w_l^2}{2g}$. Due to shock losses at the entrance to the guide-vanes a friction head, $p_l H_n$, is lost in passing from Point a to Point l. At Point a the pressure head, h_a, and the velocity head, $\frac{w_a^2}{2g}$, exist, w_a being the absolute exit velocity. Hence,

$$h_a + \frac{w_a^2}{2g} = h_l + \frac{w_l^2}{2g} + p_l H_n \dots\dots\dots\dots\dots\dots (4)$$

In order to convert as much as possible of the velocity head, $\frac{w_l^2}{2\,g}$, into pressure, the velocity, w_l, should be gradually reduced in passing from Point l through the guide-vanes. Let w_d represent the velocity and $\frac{w_d^2}{2\,g}$ the velocity head at the exit from the casing, and let $p_d\,H_n$ be the friction head expended in carrying the water through the guide-passages and casing. Then,

$$h_l + \frac{w_l^2}{2\,g} = h_d + \frac{w_d^2}{2\,g} + p_d\,H_n \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

The first form of the so-called fundamental equation is obtained by adding Equation (1) to Equation (5), which gives Equation (6). For convenience Equations (1) to (5) are here re-stated:

$$h_w = h_s + \frac{w_s}{2\,g} + p_s\,H_n$$

$$h_s + \frac{w_s^2}{2\,g} = h_e + \frac{w_e^2}{2\,g} + p_e\,H_n$$

$$h_e + \frac{v_e^2}{2\,g} + \frac{u_a^2 - u_e^2}{2\,g} = h_a + \frac{v_a^2}{2\,g} + p_a\,H_n$$

$$h_a + \frac{w_a^2}{2\,g} = h_l + \frac{w_l^2}{2\,g} + p_l\,H_n$$

$$h_l + \frac{w_l^2}{2\,g} = h_d + \frac{w_d^2}{2\,g} + p_d\,H_n$$

$$\frac{v_e^2 - w_e^2 - u_e^2 - v_a^2 + w_a^2 + u_a^2}{2\,g} = h_d - h_w$$

$$+ H_n\,(p_s + p_e + p_a + p_l + p_d) + \frac{w_d^2}{2\,g} \ldots\ldots\ldots\ldots\ldots (6)$$

From Fig. 17, it follows that,

$$h_d - h_w = H_n \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

As already explained, H_n is the desired head at the exit from the casing. The friction head necessary to overcome the losses in the delivery pipe has not been considered, because these losses may vary between wide limits and are dependent on the length and cross-section of the pipe. The velocity head, $\frac{w_d^2}{2\,g}$, at discharge from the pump is a certain fraction of the desired head, H_n. This fraction can be represented by the coefficient, α, so that,

$$\frac{w_d^2}{2\,g} = \alpha\,H_n \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

The coefficient, α, gives the magnitude of the exit losses, and varies with the desired pressure head when the velocity, w_d, is constant. This will be shown more clearly later.

In order to simplify Equation (6), let,

$$(p_s + p_e + p_a + p_l + p_d) = p \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Substituting Equations (7), (8), and (9) in Equation (6), the general form of the fundamental equation is obtained,

$$v_e^2 - w_e^2 - u_e^2 - v_a^2 + w_a^2 + u_a^2 = 2\,g\,H_n\,(1 + p + \alpha) \quad \ldots\ldots(10)$$

Let,

$$(1 + p + \alpha) = \eta \quad \ldots\ldots(11)$$

In order to obtain the total head, H_b, which is used in the calculation of centrifugal pumps and represents the theoretical head which the pump can deliver considering no losses whatever, the desired head, H_n, can be multiplied by the factor, η. Hence,

$$\eta\,H_n = H_b \quad \ldots\ldots(12)$$

From the geometric relations:

$$v_e^2 = u_e^2 + w_e^2 - 2\,u_e\,w_e \cos \delta_e \quad \ldots\ldots(13)$$

and

$$v_a^2 = u_a^2 + w_a^2 - 2\,u_a\,w_a \cos \delta_a \quad \ldots\ldots(14)$$

Substituting in Equation (10) the values of v_e^2, v_a^2, and $(1 + p + \alpha)$ from Equations (11), (12), and (13), the second form of the fundamental equation is derived,

$$u_a\,w_a \cos \delta_a - u_e\,w_e \cos \delta_e = \eta\,g\,H_n \quad \ldots\ldots(15)$$

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE HYDRAULIC DESIGN OF FLUME AND SIPHON TRANSITIONS

BY JULIAN HINDS,* M. AM. SOC. C. E.

SYNOPSIS

Considerable attention has been given by the United States Bureau of Reclamation to the hydraulic design of transitions for flumes and siphons, and rules have been established for the proportioning of important structures. A summary of these rules and of the experiences leading up to them is herewith presented. No attempt is made to present a complete treatise on transition structures, and only points on which definite experience has been gained, or opinions formed, are introduced.

Although the discussion is generally confined to inlets and outlets to and from inverted siphons and flumes, the principles established may be applied to any structure designed to change the shape or cross-sectional area of an open stream of water.

In this paper the following principal facts are shown:

1.—Unimportant transitions, where velocities are low, may be designed arbitrarily, by adaptation from successful structures operating under similar conditions.

2.—For important structures, especially those involving velocities in excess of 6 to 8 ft. per sec., careful detailed computations must be made. Extremely slender, and carefully constructed, transitions, if not proportioned in exact accordance with the hydraulic requirements, may prove seriously defective. Examples of such cases are shown, and a proposed method of computation is outlined.

NOTE.—The Special Committee on Irrigation Hydraulics selected the subject of "Losses in Canal Conversions" as one of ten for study and research. This paper was submitted to the Committee by the author, and the Committee has recommended its publication in *Proceedings* in order to elicit discussion of the subject. (See Progress Report of the Committee, *Proceedings*, Am. Soc. C. E., March, 1927, Society Affairs, p. 121). Written discussion on this paper will be closed in **February, 1928.**

* Res. Engr., The J. G. White Eng. Corporation, Pabellón, Aguascalientes, Mexico.

3.—Experimental data collected by the U. S. Bureau of Reclamation are summarized, showing the efficiencies that may be expected with various types of construction, the effects of conduit curvature on outlet efficiency, and the influence of critical flow and the hydraulic jump on the action of inlets and outlets.

Division of Subject

The subject is discussed under four main headings, as follows:

1.—Unimportant, or low-velocity structures, not involving the hydraulic jump or flow at the critical depth.
2.—Important, or high-velocity structures, not involving the hydraulic jump or flow at the critical depth.
3.—Structures involving the hydraulic jump or flow at the critical depth.
4.—Experimental data.

Notations and Abbreviations.—The notations and abbreviations used are listed for convenient reference, as follows:

A = area of water prism
Q = discharge
V = velocity
WS = water surface
ΔWS = change in water surface
hv = velocity head
Δhv = change in velocity head
B = bottom width of channel
T = width of channel at water surface
d = depth of water
H = height of lining or wall
s_f = friction slope
h_f = head lost in friction
W = width of channel at top of lining or wall

1.—The Design of Secondary Structures not Affected by Critical Flow

All structures for changing the shape or cross-sectional area of a water conduit cause disturbances in flow, which may be objectionable in themselves or in the resulting losses in head. In designing an inlet, it is necessary to provide for a drop in the water surface, sufficient to produce the required increase in velocity head and to overcome friction and entrance losses. At an outlet the water surface will rise theoretically a vertical distance equal to the reduction in velocity head. The actual rise, usually referred to as recovery of head, is less than the theoretical, because of frictional and outlet losses.

No satisfactory theory for computing the transition losses for a given structure has been proposed. The magnitude of these losses can only be determined experimentally at present. The simple considerations just outlined are often taken as sufficient for the design of a transition, the form and details being determined from precedent.

The U. S. Bureau of Reclamation, over a period of more than twenty years, has accumulated a large variety of detailed designs for transition

structures of secondary importance. The preparation of a new design for a structure of this class is usually accomplished by changing the details of a previous structure, known to be satisfactory, to suit the new conditions. A collection of possible types of simple transitions, sketched from some of these designs, is shown on Figs. 1, 2, 3, and 4. These structures differ widely in degree of perfection, and in the losses which they may be expected to produce.

The simplest forms of pipe inlets or outlets are illustrated by Types (1), (2) and (3), Fig. 1. These types are used for culverts, and are satisfactory for small pipes. It is probably reasonable to assume entrance and outlet losses equal to 0.5 and 1.0, respectively, of the velocity head in the pipe for these structures. Types (4), (5) and (6), Fig. 1, are somewhat more complicated structurally, and are perhaps slightly better hydraulically. Type (7), Fig. 1, which is sometimes used for carrying small irrigation canals under roads and railroads in cuts, is poor hydraulically, and is subject to stoppage by silt and weeds. Types (8) and (11), Fig. 1, are slightly better, but more expensive to construct. Types (9) and (10), Fig. 1, are fairly efficient, involving entrance and outlet losses, respectively, of perhaps 0.25 and 0.50 of the velocity head. Types (12) to (17), inclusive, Fig. 2, illustrate trash rack, weir, and control-gate arrangements for pipe inlets.

It is considered advantageous to affect velocity changes under pressure, where convenient, and Types (18) and (19), Fig. 2, are perhaps more efficient than the somewhat similar Types (9) and (10), Fig. 1. However, it is doubtful whether their increased cost is justified. Type (20), Fig. 2, shows a device sometimes used to check high velocities at the outlet of a pipe or culvert. Types (21) and (22), Fig. 2, show pipe ends which may be constructed without the use of forms, provided the bank around the pipe can be trimmed to the required dimensions. An inside form is required for the bellmouth in Type (21). Type (23), Fig. 3, utilizes a precast bellmouth, and requires no forming. Types (24) to (28), inclusive, Fig. 3, represent the best practice in transition structures constructed without accurately computed proportions. It is doubtful whether an outlet as elaborate as those shown on Types (27) or (28) should ever be constructed without having the proportions carefully calculated throughout. Fig 4 shows a number of flume transitions varying from a simple head-wall, Type (30), to a carefully warped structure, Type (40).

A type of structure suited to a given set of conditions may be selected from the examples in Figs. 1 to 4, the probable loss of head estimated by comparison with some structure for which the losses have been measured, and a reasonably good design prepared. The observed data given on Fig. 17 will be found helpful for estimating the losses.

All the types shown on Figs. 1, 2, 3, and 4, indicate construction in concrete, but many of them may be built of timber. The hydraulic properties of timber transitions seldom receive serious consideration, for which reason purely wooden types are not shown.

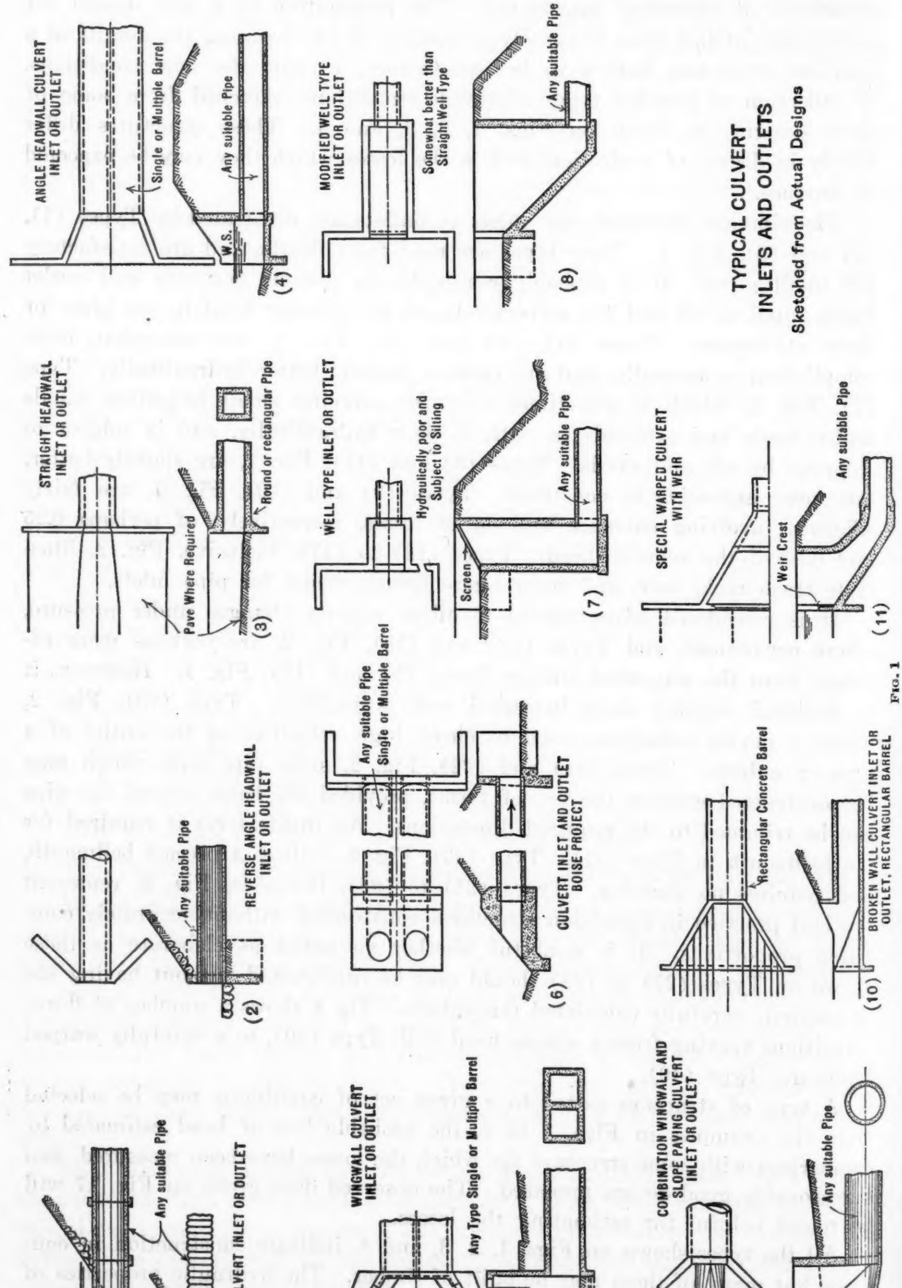

TYPICAL CULVERT
INLETS AND OUTLETS
Sketched from Actual Designs
(1) SIMPLE CULVERT INLET OR OUTLET
Any suitable Pipe
(2)
REVERSE ANGLE HEADWALL INLET OR OUTLET
Any suitable Pipe
(3)
STRAIGHT HEADWALL INLET OR OUTLET
Pave Where Required
Round or rectangular Pipe
(4)
ANGLE HEADWALL CULVERT INLET OR OUTLET
Single or Multiple Barrel
Any suitable Pipe
W.S.
(5)
WINGWALL CULVERT INLET OR OUTLET
Any Type Single or Multiple Barrel
(6)
CULVERT INLET AND OUTLET BOISE PROJECT
Any suitable Pipe
Single or Multiple Barrel
(7)
WELL TYPE INLET OR OUTLET
Hydraulically poor and Subject to Silting
Any suitable Pipe
Screen
(8)
MODIFIED WELL TYPE INLET OR OUTLET
Somewhat better than Straight Well Type
Any suitable Pipe
(9)
COMBINATION WINGWALL AND SLOPE PAVING CULVERT INLET OR OUTLET
Any suitable Pipe
(10)
BROKEN WALL CULVERT INLET OR OUTLET, RECTANGULAR BARREL
Rectangular Concrete Barrel
(11)
SPECIAL WARPED CULVERT WITH WEIR
Weir Crest
Any suitable Pipe

FIG. 1

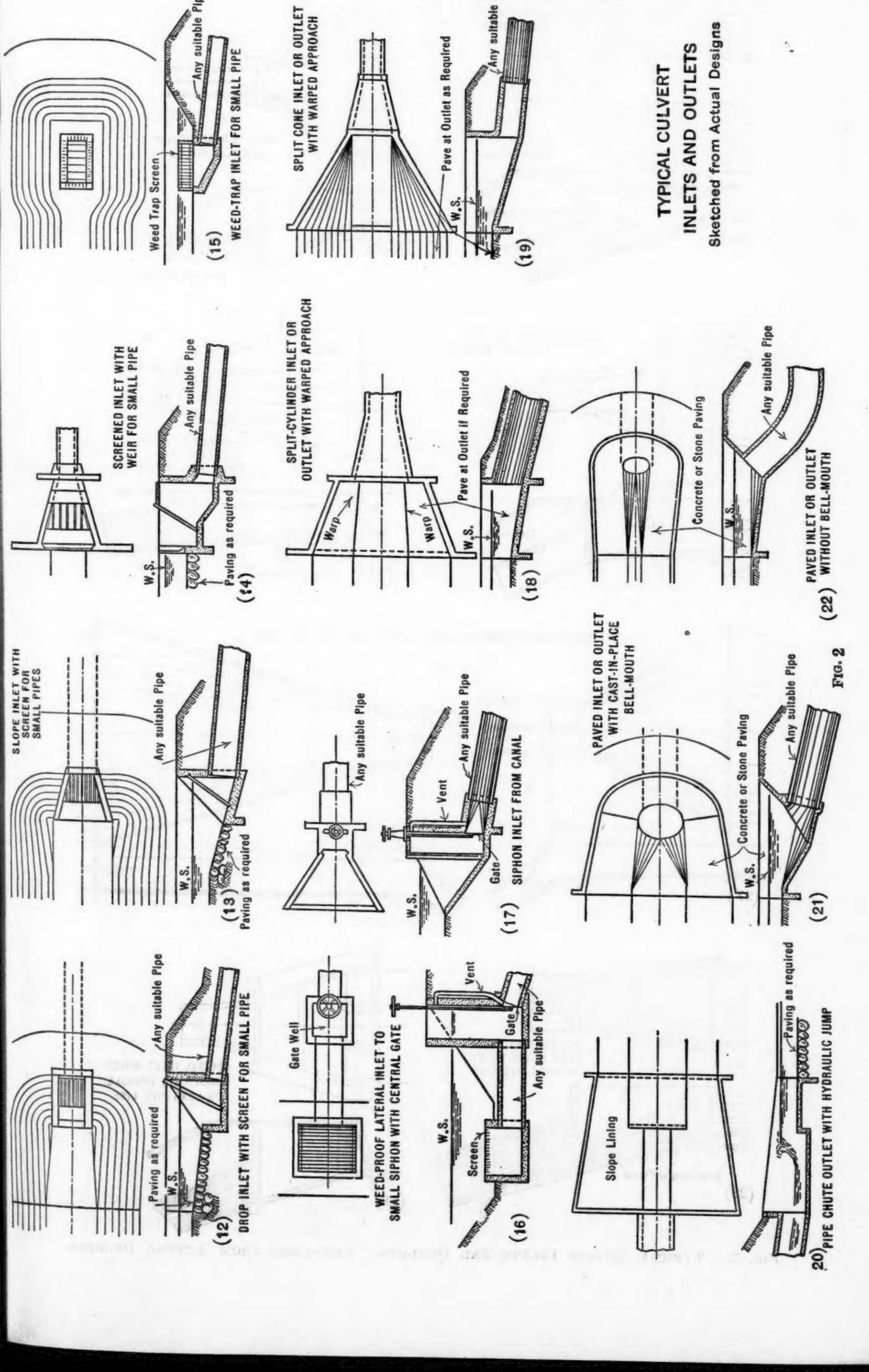

TYPICAL CULVERT
INLETS AND OUTLETS
Sketched from Actual Designs
Paving as required
W.S.
Any suitable Pipe
(12) DROP INLET WITH SCREEN FOR SMALL PIPE
SLOPE INLET WITH SCREEN FOR SMALL PIPES
(13) Paving as required
SCREENED INLET WITH WEIR FOR SMALL PIPE
(14) Paving as required
Weed Trap Screen
(15) WEED-TRAP INLET FOR SMALL PIPE
Gate Well
WEED-PROOF LATERAL INLET TO SMALL SIPHON WITH CENTRAL GATE
Screen
Vent
Gate
(16)
Vent
Gate
(17) SIPHON INLET FROM CANAL
SPLIT-CYLINDER INLET OR OUTLET WITH WARPED APPROACH
Warp
Pave at Outlet if Required
(18)
SPLIT CONE INLET OR OUTLET WITH WARPED APPROACH
Pave at Outlet as Required
(19)
Slope Lining
(20) PIPE CHUTE OUTLET WITH HYDRAULIC JUMP
PAVED INLET OR OUTLET WITH CAST-IN-PLACE BELL-MOUTH
Concrete or Stone Paving
(21)
(22) PAVED INLET OR OUTLET WITHOUT BELL-MOUTH

FIG. 2

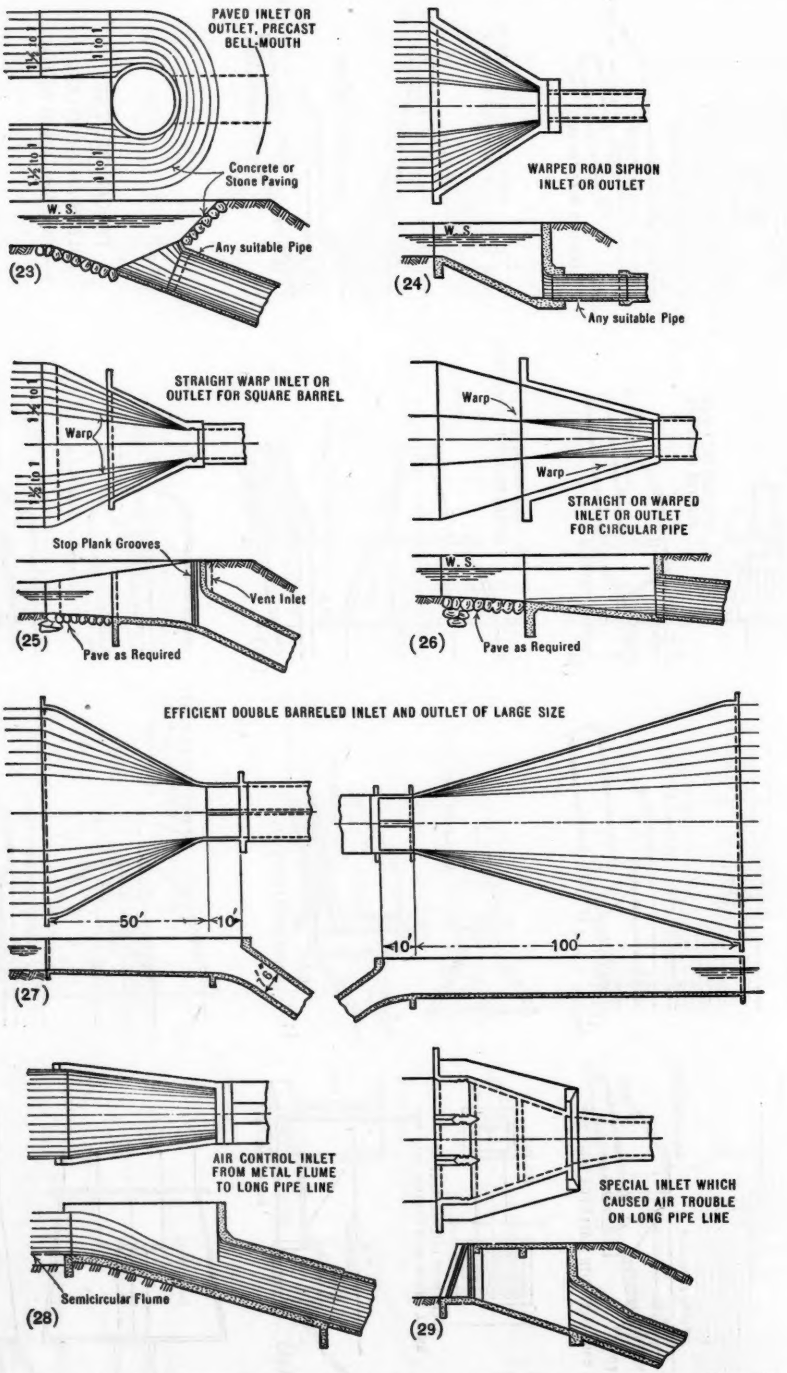


FIG. 3.—TYPICAL SIPHON INLETS AND OUTLETS. SKETCHES FROM ACTUAL DESIGNS.

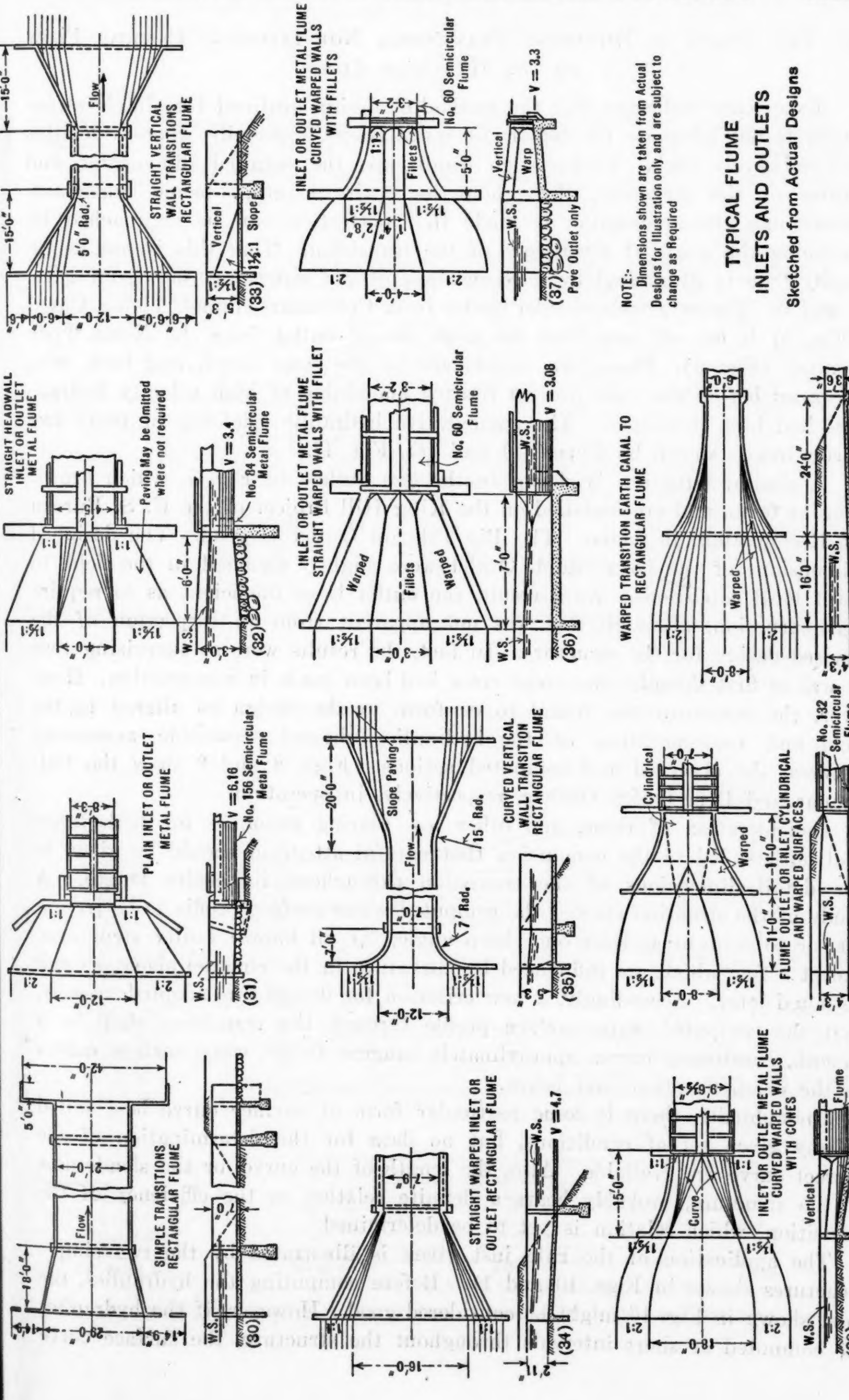

STRAIGHT VERTICAL WALL TRANSITIONS RECTANGULAR FLUME
INLET OR OUTLET METAL FLUME CURVED WARPED WALLS WITH FILLETS
No. 60 Semicircular Flume
V = 3.3
Pave Outlet only
NOTE:- Dimensions shown are taken from Actual Designs for Illustration only and are subject to change as Required
TYPICAL FLUME
INLETS AND OUTLETS
Sketched from Actual Designs
STRAIGHT HEADWALL INLET OR OUTLET METAL FLUME
Paving May be Omitted where not required
No. 84 Semicircular Metal Flume
V = 3.4
INLET OR OUTLET METAL FLUME STRAIGHT WARPED WALLS WITH FILLET
V = 3.08
WARPED TRANSITION EARTH CANAL TO RECTANGULAR FLUME
Pave if Required
PLAIN INLET OR OUTLET METAL FLUME
No. 156 Semicircular Metal Flume
V = 6.16
CURVED VERTICAL WALL TRANSITION RECTANGULAR FLUME
Slope Paving
FLUME OUTLET OR INLET CYLINDRICAL AND WARPED SURFACES
No. 132 Semicircular Flume
SIMPLE TRANSITIONS RECTANGULAR FLUME
STRAIGHT WARPED INLET OR OUTLET RECTANGULAR FLUME
V = 4.7
INLET OR OUTLET METAL FLUME CURVED WARPED WALLS WITH CONES

FIG. 4

2.—The Design of Important Transitions, Not Involving Critical Flow or the Hydraulic Jump

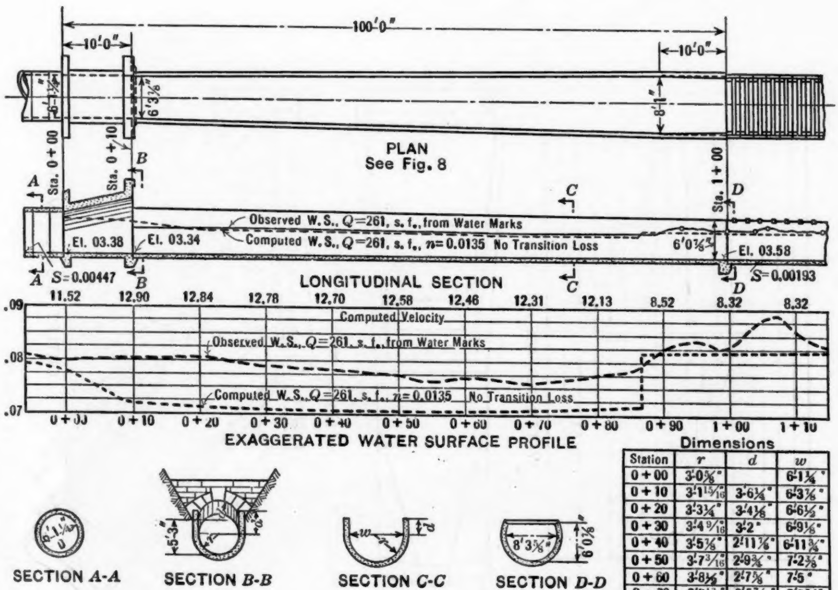
Experience indicates that the method of design outlined for simple structures is not adequate for important installations, especially where velocities are relatively high. Under such conditions, the detailed dimensions and forms of the structure, throughout its entire length, become important. Sometimes the assumption is made that perfection can be approached by reducing the angle of divergence of the transition. That this is not necessarily true is illustrated by a comparison of the structures shown on Figs. 5 and 6. The extremely slender outlet from Columnar Tunnel, Tieton Canal (Fig. 5) is less efficient than the more abrupt outlet from the North Fork Tunnel (Fig. 6). These two outlets are on the same canal, and both were designed before even the present limited knowledge of high velocity hydraulics had been developed. The comparative hydraulic efficiency of these two structures is shown by Types (*r*) and (*t*), Fig. 17.

A similar situation is shown in the two outlets in Fig. 7, which are of similar form, and are installed on the King Hill Project of the U. S. Bureau of Reclamation, in Idaho. The Big Pilgrim Outlet is good. The designed dimensions of the Deer Gulch Outlet were slightly changed in the field, to meet local conditions. As a result, the outlet is so inefficient as to require reconstruction, although this was not apparent from an inspection of the revised design for the structure. In fact, the results were so surprising that it was at first thought that some error had been made in construction. However, the structure was found to conform to the design as altered in the field and recomputation of the hydraulics showed reasonable agreement between the observed and computed actions. Figs. 8 and 9 show the Columnar and Deer Gulch Outlets, respectively, in operation.

Consideration of these, and other less glaring examples for both inlets and outlets, led to the conclusion that careful attention should be given to the detail dimensions of the transition throughout its entire length. A study of the situation showed the computed water-surface profile to be irregular, or to contain at least one sharp angle, for all known faulty structures, except a few which are influenced by curvature in the channel above, as will be noted later. Accordingly, a new criterion for design was adopted, namely, that the computed water-surface profile through the transition shall be a smooth, continuous curve, approximately tangent to the water-surface curves in the channels above and below.

Undoubtedly, there is some particular form of surface curve best suited to any given set of conditions, but no data for the determination of the correct curve are available. Also, the length of the curve, or the slenderness of the structure, probably bears a definite relation to the efficiency of the transition, which relation is yet to be determined.

The application of the rule just given is illustrated by the two simple structures shown in Figs. 10 and 11. Before computing the hydraulics, the inlet shown in Fig. 10 might be considered good. However, if the hydraulics are computed at short intervals throughout the structure, the surface curve



Dimensions

Station	r	d	w
0 + 00	3'-0⅝"		6'-1¼"
0 + 10	3'-1 15/16"	3'-6¼"	6'-3⅞"
0 + 20	3'-3¼"	3'-4⅛"	6'-6½"
0 + 30	3'-4 9/16"	3'-2"	6'-9⅛"
0 + 40	3'-5⅞"	2'-11⅞"	6'-11¾"
0 + 50	3'-7 3/16"	2'-9¾"	7'-2⅜"
0 + 60	3'-8½"	2'-7⅝"	7'-5"
0 + 70	3'-9 13/16"	2'-5 7/16"	7'-7⅝"
0 + 80	3'-11⅛"	2'-3 5/16"	7'-10¼"
0 + 90	4'-0½"	2'-1 3/16"	8'-1"
1 + 00	4'-1 13/16"	1'-11 1/16"	8'-3⅝"

FIG. 5.—PLAN AND CROSS-SECTIONS OF OUTLET, COLUMNAR TUNNEL, TIETON CANAL.

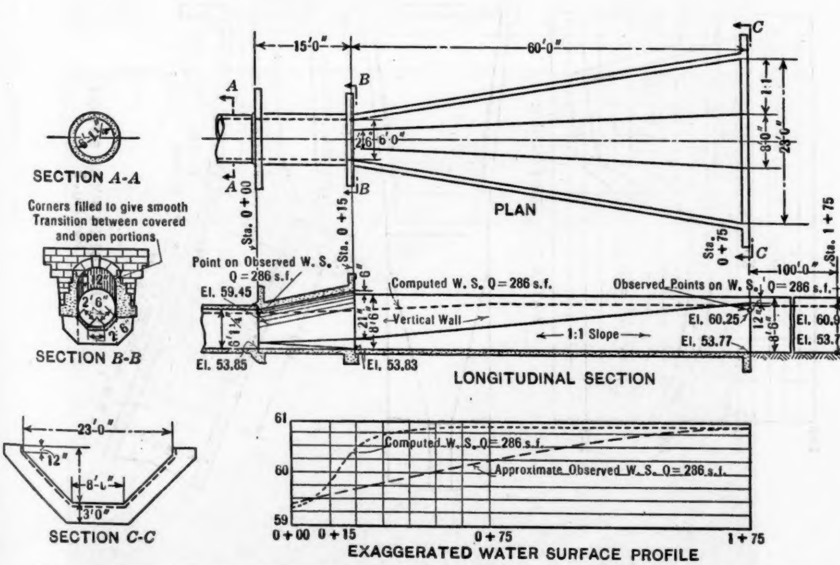


FIG. 6.—PLAN AND CROSS-SECTIONS OF OUTLET, NORTH FORK TUNNEL, TIETON CANAL.

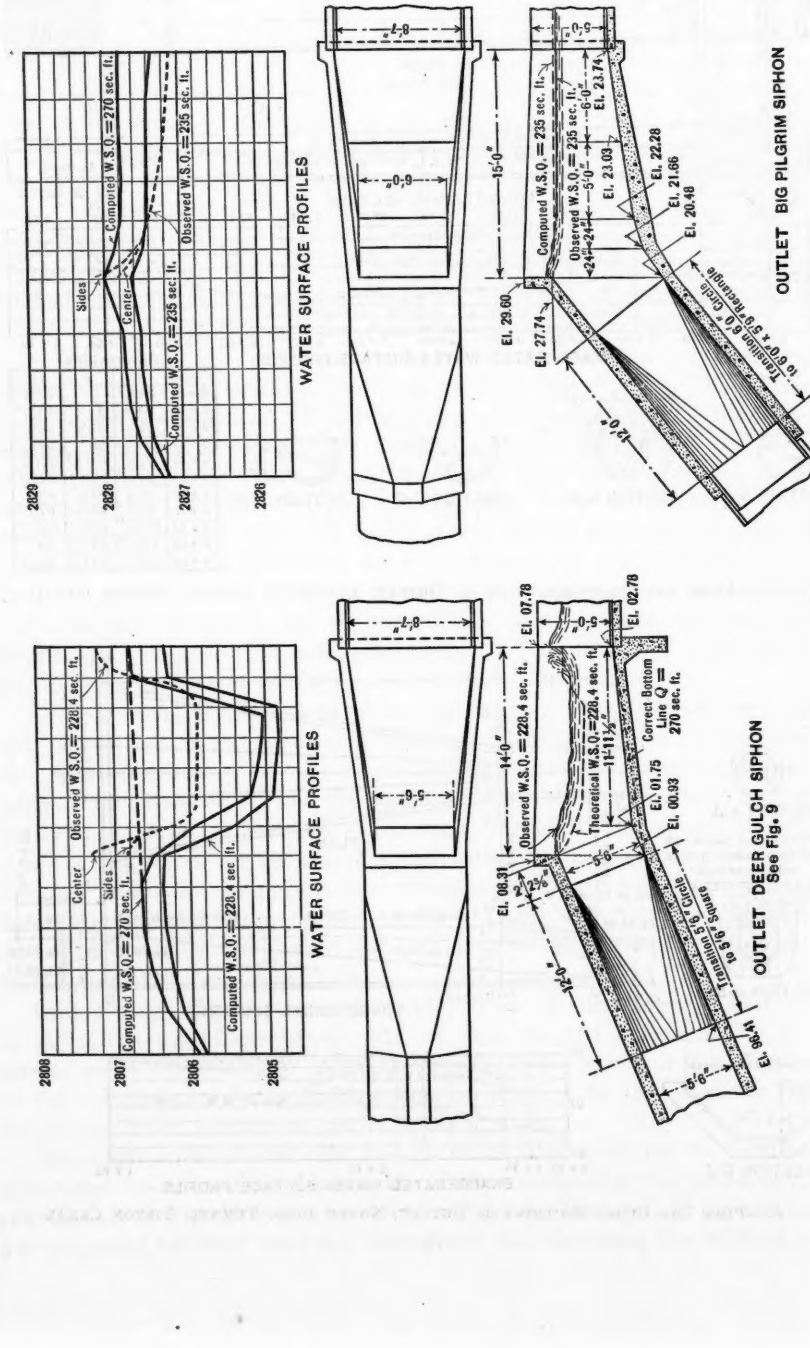


FIG. 7.—GOOD AND POOR SIPHON OUTLETS OF SIMILAR DESIGN.

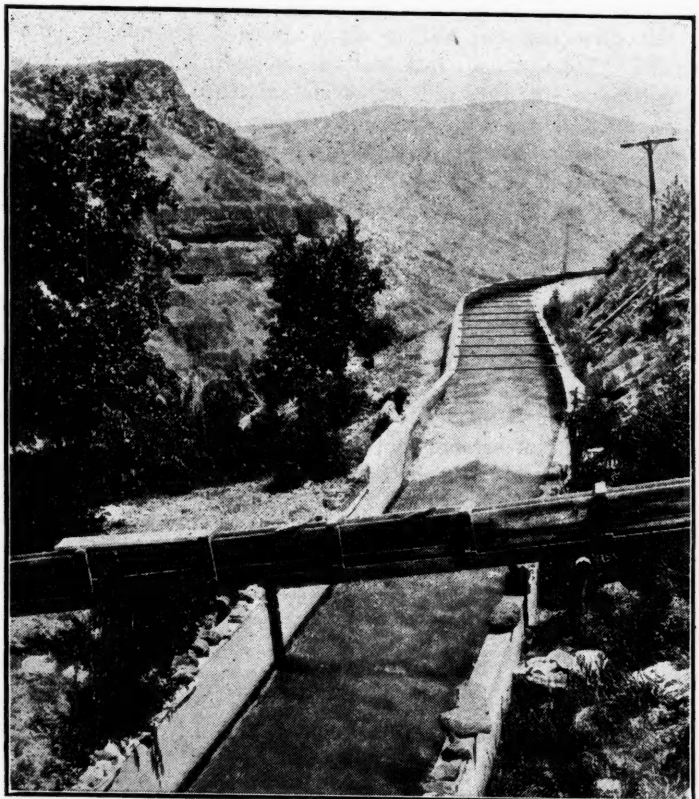
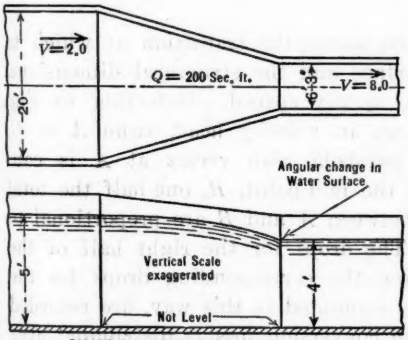


FIG. 8.—VIEW OF OUTLET, COLUMNAR TUNNEL.

FIG. 9.—VIEW OF OUTLET, DEEP GULCH SIPHON.

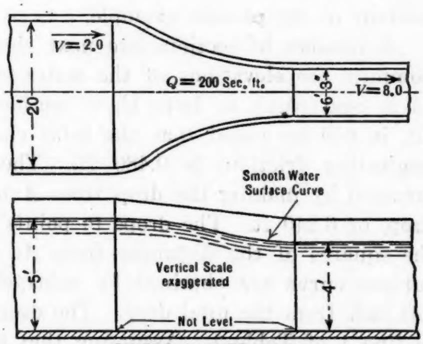
will be found to contain a sharp angle at the junction with the narrow channel, and marked disturbances in flow may be expected. By properly curving the walls of the inlet, as shown in Fig. 11, this condition can be avoided, the rate of change in acceleration being changed in such a way that the water-surface profile becomes a smooth, continuous curve.

A plan based on the principles illustrated in Fig. 11 is used by the U. S. Bureau of Reclamation for all important transitions. If desired, the dimensions of the structure may be assumed and the surface curve computed from Bernoulli's theorem, the dimensions being subsequently changed and the curve recalculated until satisfactory results are secured. A more direct solution is obtained if the water-surface curve is first determined and the dimensions of the structure are computed to conform. The procedure recommended will be illustrated by examples of actual designs. Before attempting to prepare designs similar to these examples the discussion of experimental data should be reviewed, especially that part which relates to the effect of curvature in flumes and siphons on the action of their outlet structures.



SIMPLE INLET--STRAIGHT TAPER

FIG. 10.



SIMPLE CURVED INLET

FIG. 11.

Example of a Flume Inlet.—Referring to Fig. 12, let it be required to design an inlet from an earth canal with a bottom width of 18 ft., side slopes of 2 : 1, to a rectangular concrete flume, 12 ft. 6 in. wide. The hydraulic properties of the canal and the flume may be assumed to be known, as shown on the diagram.

It is desirable first to determine approximately the length that will be required for the transition. In the absence of specific knowledge this must be done arbitrarily. For structures of the type under consideration the U. S. Bureau of Reclamation has adopted the rule of making the length such that a straight line joining the flow line at the two ends of the transition, as *D-E*, Fig. 12, will make an angle of about 12½° with the axis of the structure. In the example shown the resulting length is 50 ft.

From information given in the discussion of experimental data, it appears that, for a structure of the type contemplated, the entrance loss may be safely assumed as 0.1 the change in velocity head, making 1.10 $\Delta h v$, the total drop to be provided, plus the drop necessary to overcome friction. If the water-

surface elevation in the canal at the upper end of the inlet is known, and if friction is temporarily neglected, the water-surface elevation in the flume at the lower end of the transition can be found, and the two end points, *A* and *C*, of the water surface profile, *A B C*, Fig. 12, determined. Between *A* and *C* the water surface theoretically may be made to follow any desired profile, within reasonable limits, the profile being controlled by the shape and size of the transition structure.

If the flow is to be smooth and the structure efficient, the theoretical water surface must be free from angles or sharp curves. The particular curve that fits the given conditions better than any other not being known, any smooth curve tangent to the normal water surface in the canal, at *A*, and in the flume, at *C*, may be used. It may be drawn arbitrarily or computed. In Fig. 12, the water surface, neglecting friction, is taken as two equal parabolas, horizontal, respectively, at *A* and *C*, and tangent at *B*. Strictly, the parabolas should be tangent to the water-surface slopes in the canal and the flume, but the small divergence of these slopes from the horizontal is unimportant in the present example.

A number of sections are next chosen across the transition at which to compute the elevations of the water surface and the structural dimensions. It is convenient to have these sections equally spaced. Referring to Fig. 12, it will be noted that the total change in velocity head, from *A* to *C*, neglecting friction, is 0.480 ft. The parabola with vertex at *A* is constructed by making the drop from *A* to the mid-point, *B*, one-half the total drop, or 0.240 ft. The drops to points between *A* and *B* are proportional to the squares of the distances from *A*. The drops for the right half of the surface curve are obtained by subtracting the corresponding drops for the left half from the total drop. The drops, computed in this way, are recorded in line 1 of Table 1. Assuming that the conversion loss is distributed over the entire length of the transition, in proportion to the change in velocity head, values of $\Delta\, h\, v$ are obtained by dividing the computed values of $\Delta\, W\, S$ by 1.10. The velocity head is obtained by adding $\Delta\, h\, v$ to the velocity head at *A*. The velocities corresponding to the assumed water-surface curve are then determined, as recorded in Line 4, Table 1, and the required area of cross-section is computed as shown in Line 5. Except that no allowance has been made for friction, this completes the hydraulic design, the area of the section at each point of sub-division being known. It remains, however, to choose shapes for the various sections, such that they will fit with each other to produce a structure of pleasing appearance, free from angles and sharp curves. The work from this point depends largely on the skill and experience of the designer. Sections may be either arbitrarily chosen and platted until satisfactory results are obtained, or the problem may be attacked systematically.

A convenient start may be made from arbitrarily sketched plans of the water line, shown dashed in Fig. 12, and of the intersection of the side slope and the bottom. The proper trial shape of these plans is a matter of judgment. The maximum angle of divergence between the water line and the

TABLE 1.—COMPUTATIONS FOR FLUME INLET.

In flume, $V = 5.98$ $h\,v = 0.553$ Elevation of water surface at $0 + 00 = 57.41$
In canal, $V = 2.75$ $h\,v = 0.117$ Entrance loss $= 0.1\,\Delta\,h\,v$

$\Delta\,h\,v = 0.436$ Water surface reversed parabola.

Line.	Item.	STATION.										
		0 + 00.	0 + 05.	0 + 10.	0 + 15.	0 + 20.	0 + 25.	0 + 30.	0 + 35.	0 + 40.	0 + 45.	0 + 50.
1	Δ *W. S.* = Drop in *W. S.**	....	0.010	0.038	0.086	0.154	0.240	0.326	0.394	0.442	0.470	0.480
2	$\Delta\,h\,v = \Delta$ *W. S.* ÷ 1.1	....	0.009	0.035	0.079	0.140	0.218	0.296	0.357	0.401	0.427	0.436
3	$h\,v = 0.117 + \Delta\,h\,v$	....	0.126	0.152	0.196	0.257	0.335	0.413	0.474	0.518	0.544	0.553
4	V	2.75	2.85	3.13	3.55	4.07	4.64	5.15	5.52	5.77	5.91	5.97
5	Area = $Q \div V$	114.40	110.50	100.60	88.75	77.40	67.88	61.20	57.10	54.60	53.30	52.70
6	0.5 T = Half width at *W. S.*	17.600	17.000	15.427	13.460	11.228	9.139	7.717	6.847	6.458	6.315	6.25
7	0.5 B = Half bottom width	9.000	8.625	7.917	7.250	6.958	6.771	6.667	6.563	6.458	6.315	6.25
8	0.5 T + 0.5 B = Average width	26.600	25.625	23.344	20.710	18.186	15.910	14.384	13.410	12.916	12.630	12.500
9	d = Area ÷ Ave. width	4.30	4.309	4.310	4.280	4.260	4.264	4.252	4.253	4.225	4.220	4.220
10	s_f = Friction slope	0.00015	0.00017	0.00020	0.00026	0.00034	0.00046	0.00061	0.00076	0.00083	0.00087	0.00090
11	$h_f = 5\,s_f$ (Use Ave. s_f)	....	0.00080	0.00090	0.00115	0.00150	0.00200	0.00270	0.00345	0.00400	0.00425	0.00445
12	$\Sigma\,h_f$	....	0.00080	0.00170	0.00285	0.00435	0.00635	0.00905	0.01250	0.01650	0.02075	0.02520
13	*W. S.* Elev. = 57.41 − Δ *W. S.* − $\Sigma\,h_f$	57.410	57.399	57.370	57.321	57.252	57.164	57.075	57.003	56.951	56.919	56.905
14	Grade = *W. S.* Elev. − d	53.110	53.090	53.060	53.041	52.992	52.900	52.823	52.750	52.726	52.699	52.685
15	0.5 T − 0.5 B	8.600	8.375	7.510	6.210	4.270	2.368	1.050	0.284	....	....	....
16	Side slopes	2.000	1.945	1.744	1.447	1.000	0.554	0.247	0.067	....	....	....
17	H = Height of lining	5.330	5.295	5.270	5.234	5.228	5.265	5.287	5.305	5.274	5.236	5.205
18	0.5 W − 0.5 B = Side slope × H	10.660	10.310	9.210	7.575	5.228	2.920	1.305	0.354	....	....	....
19	0.5 W = 0.5 Top width	19.660	18.935	17.127	14.825	12.186	9.691	7.972	6.917	6.458	6.315	6.250
20	0.5 W to nearest ⅛ in.	19 ft. 8 in.	18 ft. 11 in.	17 ft. 1½ in.	14 ft. 10 in.	12 ft. 2 in.	9 ft. 8⅛ in.	7 ft. 11⅝ in.	6 ft. 11 in.	6 ft. 5½ in.	6 ft. 4 in.	6 ft. 3 in.

* Neglecting friction temporarily.

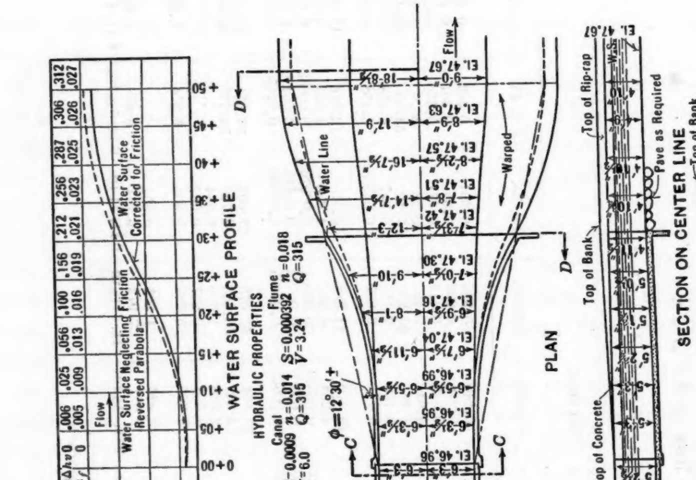


FIG. 13.—TYPICAL RECTANGULAR FLUME OUTLET.

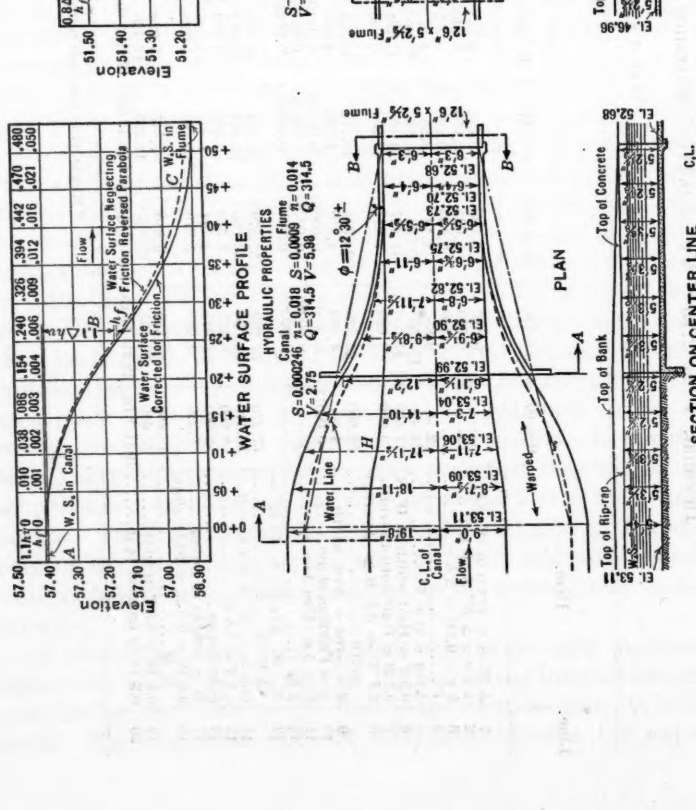


FIG. 12.—TYPICAL RECTANGULAR FLUME INLET.

axis of the structure should not be too great. A limit of 25° is recommended. These two plans having been drawn, the half top widths and half bottom widths are scaled, as recorded in Lines 6 and 7, Table 1. The sum of these half widths gives the average width from which the depth of water may be computed.

Having determined the velocity, and the shape and depth of the channel, the friction slope, s_f, and the accumulated friction head, h_f, for each point may be computed. The water surface profile may then be corrected for friction, and the profile of the bottom of the channel computed and platted. This latter profile should be free from objectionable irregularities. Otherwise, the assumed plan may be changed as required, to increase or decrease the depth at points of irregularity. If it is not possible to secure regularity in both the plan and the bottom profile, the assumed water-surface profile may be altered. A slight change in the elevation of the water surface at a given point usually makes an appreciable change in the dimensions of the structure.

It is advisable to plat a preliminary bottom profile before computing the friction slope. The friction makes very little change in the form of the bottom profile, and labor is saved by leaving it until the profile is otherwise satisfactory. In fact, the frictional losses may often be entirely ignored without serious error.

The last six lines of Table 1 relate to the structural dimensions of the transition, and are self-explanatory. Figures for these lines need be inserted only for the design finally adopted.

Example of a Flume Outlet.—An outlet transition from a flume is designed in the same way as an inlet, the only essential difference being that the conversion loss is subtracted from $\Delta\, h\, v$ to obtain $\Delta\, W\, S$. A typical design for an outlet from a rectangular flume to an earth canal is shown in Fig. 13. The computations are shown in Table 2.

In the example shown, the length of the outlet structure is determined on the same basis as the length of the inlet structure. It is now generally conceded that for equal efficiency an inlet may be made shorter than an outlet, for the same velocity change, and it is hoped that the discussion of the subject will bring out definite rules for determining the proper length of each type of structure. The experience of the U. S. Bureau of Reclamation indicates that for properly designed outlets of the type shown in Fig. 13, a length as determined by making $\phi = 12\frac{1}{2}°$ is sufficient for ordinary purposes. Inlets are generally made the same length because: (*a*) sharper warps are difficult to construct; (*b*) short transitions do not afford secure anchorage to the canal; (*c*) using the same length makes the forms interchangeable; and (*d*) an average divergence of $12\frac{1}{2}°$ yields a structure of pleasing appearance and reasonable cost.

Table 2 is similar to Table 1. It will be noted that an allowance of $0.2\,\Delta\, h\, v$ is made for outlet losses, which is somewhat more than might be expected from existing experimental data. Any excess in possible recovery of head over the computed recovery affords a small margin of safety against

TABLE 2.—Computations for Flume Outlet.

In flume, $V = 5.98$ $h\,v = 0.553$ Elevation of water surface at $0 + 00 = 51.18$
In canal, $V = 3.24$ $h\,v = 0.163$ Outlet loss $= 0.2\,\Delta\,h\,v$

$\Delta\,h\,v = 0.390$ Water surface reversed parabola.

Line.	Item.	Station. 0 + 00	0 + 05	0 + 10	0 + 15	0 + 20	0 + 25	0 + 30	0 + 35	0 + 40	0 + 45	0 + 50
1	Δ *W. S.* = Rise in *W. S.**...	....	0.006	0.025	0.056	0.100	0.156	0.212	0.256	0.287	0.306	0.312
2	$\Delta\,hv = \Delta$ *W. S.* $\div$ 0.80.....	....	0.008	0.031	0.070	0.125	0.195	0.265	0.320	0.359	0.382	0.390
3	$h\,v = 0.553 - \Delta\,h\,v$.........	0.553	0.545	0.522	0.483	0.428	0.358	0.228	0.233	0.194	0.171	0.163
4	V........................	5.97	5.92	5.80	5.57	5.25	4.80	4.30	3.87	3.53	3.32	3.24
5	Area = $Q \div V$.............	52.78	53.25	54.30	56.55	60.00	65.62	73.30	81.40	89.20	94.90	97.22
6	0.5 T = Half width at *W. S.*	6.250	6.292	6.458	6.890	7.824	9.300	11.232	13.210	14.800	16.010	16.600
7	0.5 B = Half bottom width.	6.250	6.292	6.458	6.628	6.792	7.042	7.292	7.667	8.208	8.750	9.000
8	0.5 T + 0.5 B = Average width..................	12.500	12.584	12.916	13.515	14.616	16.342	18.524	20.877	23.008	24.760	25.600
9	d = Area $\div$ Ave. width	4.220	4.232	4.202	4.182	4.104	4.018	3.955	3.900	3.876	3.835	3.800
10	s_f = Friction slope.........	0.00090	0.00088	0.00085	0.00075	0.00063	0.00051	0.00041	0.00033	0.00027	0.00024	0.00023
11	$h_f = 5\,s_f$ (Using Ave. s_f)....	....	0.005	0.004	0.004	0.003	0.003	0.002	0.002	0.002	0.001	0.001
12	$\Sigma\,h_f$........................	....	0.005	0.009	0.013	0.016	0.019	0.021	0.023	0.025	0.026	0.027
13	*W. S.* Elev. = 51.18 + Δ *W. S.* $- \Sigma\,h_f$...........	51.180	51.181	51.196	51.223	51.264	51.317	51.371	51.413	51.442	51.460	51.465
14	Grade = *W. S.* Elev. $- d$...	46.960	46.949	46.994	47.041	47.160	47.299	47.416	47.513	47.566	47.625	47.665
15	0.5 T − 0.5 B...............	....	....	....	0.262	1.032	2.258	3.940	5.543	6.692	7.260	7.600
16	Side slopes.................	....	....	....	0.0625	0.247	0.553	0.997	1.422	1.727	1.892	2.000
17	H = Height of lining.......	5.210	5.255	5.244	5.231	5.146	5.041	4.958	4.895	4.876	4.751	4.845
18	0.5 W − 0.5 B = Side slope × H.....................	....	....	....	0.326	1.270	2.790	4.940	6.960	8.420	9.000	9.690
19	0.5 W = 0.5 Top width......	6.250	6.292	6.458	6.955	8.062	9.832	12.232	14.627	16.628	17.750	18.690
20	0.5 W to nearest ½ in.......	6 ft. 3 in.	6 ft. 3½ in.	6 ft. 5½ in.	6 ft. 11½ in.	8 ft. 1 in.	9 ft. 10 in.	12 ft. 3 in.	14 ft. 7½ in.	16 ft. 7½ in.	17 ft. 9 in.	18 ft. 8½ in.

* Neglecting friction temporarily.

a reduction of free-board due to fouling of the canal below the flume. If there is any doubt as to the maintenance of a clean channel below the flume a greater allowance for outlet loss should be made. In extreme cases it may be desirable to assume no recovery at flume outlets, even for structures of careful design. The full actual recovering capacity of the outlet is thus made available for raising the canal water surface above normal without encroaching on the computed free-board in the flume.

Where an allowance is made for a greater outlet loss than is actually necessary, the destruction of the excess energy must be considered. If the flume velocity is relatively low, the depth being greater than the critical depth, a recovery will actually occur at the outlet, whether provided for or not. The velocity in the flume will be increased because of the excess head, and the depth decreased, making recovery necessary. If flow in the flume is at or below the critical depth no draw-down above the outlet is possible, and the excess energy must be dissipated in waves and eddies, or by a hydraulic jump, in the canal or in the outlet. The resulting eddies may cause the canal to erode.

Example of a Siphon Inlet.—The design of a transition from an earth canal with a 12-ft. bottom width to a siphon 7 ft. in diameter is illustrated in Fig. 14 and Table 3. The procedure shown differs from that recommended for flume inlets in two particulars. Part of the change in velocity is made under cover, in the transition from square to circular conduit. Any attempt to control minutely the form of this part of the transition results in construction difficulties. The transition, therefore, is usually computed only up to the head-wall. Since the velocity is still increasing at the head-wall, it is not desirable to use two equal tangent parabolas for the surface profile through the inlet. The part of the surface curve to the right of the point, *J*, on the water surface profile, Fig. 14, is fixed. The curve from *I* to *J* should be drawn approximately tangent to it. However, certain other necessary irregularities at *J* make exact tangency unimportant, and the practice has been established of assuming a single parabola, with a vertex at *I*, passing through the computed water surface at *J*. The parabola is chosen for convenience, and not because of any supposed superiority over other forms of curves. The drop from *I* to *J* is taken as $1.1\,\Delta h v$, the velocity at *J* being computed from the area of the square covered conduit.

It is considered desirable to have the top of the siphon barrel at the head-wall of the transition slightly submerged at full flow. This is thought to minimize reduction in capacity due to the introduction of air into the siphon. It is the practice of the U. S. Bureau of Reclamation to place the theoretical corner at the intersection of the head-wall and the top of the siphon barrel $1.5\,\Delta h v$ below the water level at the up-stream end of the inlet, with a minimum of 18 in. This practice makes it impracticable to construct the lower end of the transition strictly in accordance with the computations. Under this condition sections just inside and just outside of the siphon barrel cannot be made hydraulically equivalent. The computations in Table 3 are

TABLE 3.—COMPUTATIONS FOR SIPHON INLET.

In siphon, $V = 4.25$ $h\,v = 0.280$ Elevation at water surface at $0 + 00 = 95.82$.
In canal, $V = 2.31$ $h\,v = 0.083$ Entrance loss $= 0.1\ \Delta\,h\,v$.

$\Delta\,h\,v = 0.197$ Water surface, single parabola.

Line.	Item.	STATION. 0 + 00	0 + 05	0 + 10	0 + 15	0 + 20	0 + 25	0 + 30	0 + 35	0 + 40†	0 + 45†
1	Δ *W. S.* = Drop in *W. S.**	......	0.003	0.011	0.024	0.043	0.067	0.096	0.131	0.172	0.217
2	$\Delta\,h\,v$ = *W. S.* ÷ 1.1	......	0.002	0.010	0.022	0.039	0.061	0.088	0.119	0.156	0.197
3	$h\,v = 0.083 + \Delta\,h\,v$	0.083	0.085	0.093	0.105	0.122	0.143	0.171	0.202	0.239	0.280
4	*V*	2.310	2.34	2.44	2.60	2.80	3.03	3.32	3.60	3.92	4.24
5	Area = *Q* ÷ *V*	91.52	89.00	85.10	80.10	74.20	68.60	62.80	57.70	53.00	49.00
6	0.5 *T* = Half width at *W. S.*	14.800	14.200	13.050	11.600	9.800	7.650	5.950	4.550	3.800	3.500
7	0.5 *B* = Half bottom width	6.000	5.950	5.750	5.400	4.900	4.350	4.000	3.600	3.500	3.500
8	0.5 *T* + 0.5 *B* = Ave. width	20.800	20.150	18.800	17.000	14.700	12.100	9.950	8.120	7.300	7.000
9	*d* = Area ÷ Ave. width	4.400	4.415	4.545	4.710	5.045	5.670	6.310	7.080	7.260	7.000
10	s_f = Friction slope	0.00011	0.00012	0.00013	0.00014	0.00016	0.00018	0.00022	0.00030	0.00045	0.00074
11	$h_f = 5\,s_f$ (Using Ave. s_f)	....	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.002	0.003
12	$\Sigma\,h_f$	....	0.001	0.002	0.003	0.004	0.005	0.006	0.007	0.009	0.012
13	*W. S.* Elev. = 95.82 − Δ *W. S.* − $\Sigma\,h_f$	95.820	95.815	95.807	95.793	95.773	95.748	95.717	95.682	95.639	95.591
14	Grade = *W. S.* Elev. − *d*	91.420	91.400	91.262	91.083	90.728	90.078	89.407	88.602	88.379	88.591
15	0.5 *T* − 0.5 *B*	8.800	8.250	7.300	6.200	4.900	3.300	1.950	0.950	0.300	....
16	Side slopes	2.000	1.870	1.600	1.316	0.970	0.582	0.309	0.134	0.038	....
17	*H* = Height of lining	5.790	5.800	5.950	6.130	6.480	7.370	8.280	9.320	10.560	11.800
18	0.5 *W* − 0.5 *B* = Side slope × *H*	11.580	10.850	9.520	8.060	6.280	4.280	2.560	1.250	0.402	....
19	0.5 *W* = 0.5 Top width	17.580	16.800	15.270	13.460	11.180	8.630	6.560	4.850	3.900	3.500
20	0.5 *W* to nearest ½ in.	17 ft. 7 in.	16 ft. 10 in.	15 ft. 3½ in.	13 ft. 5½ in.	11 ft. 2 in.	8 ft. 7½ in.	6 ft. 6½ in.	4 ft. 10 in.	3 ft. 11 in.	3 ft. 6 in.

* Neglecting friction temporarily.
† Grade elevations at Stations 0 + 40 and 0 + 45 are arbitrarily changed to 87.60 and 86.60 to obtain a smooth grade curve.

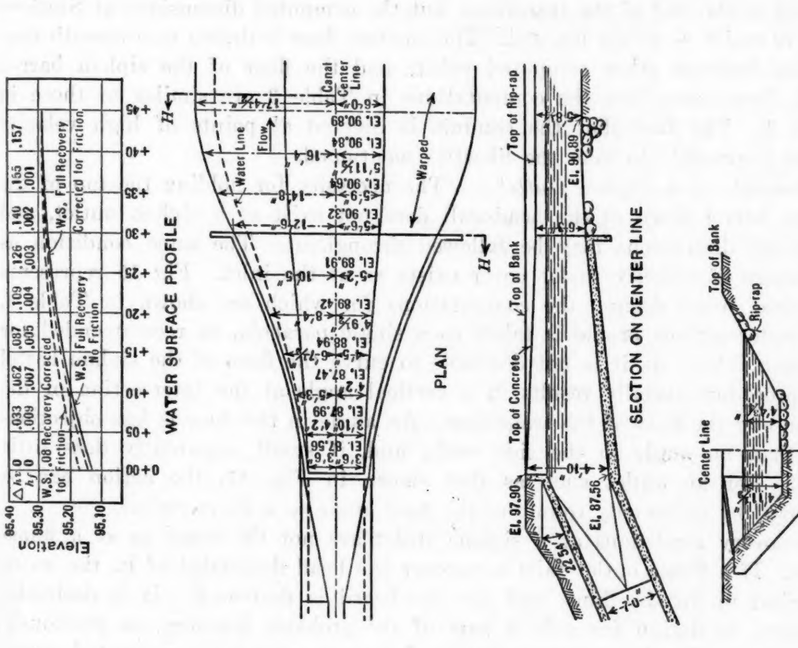


FIG. 15.—TYPICAL CONCRETE SIPHON OUTLET.

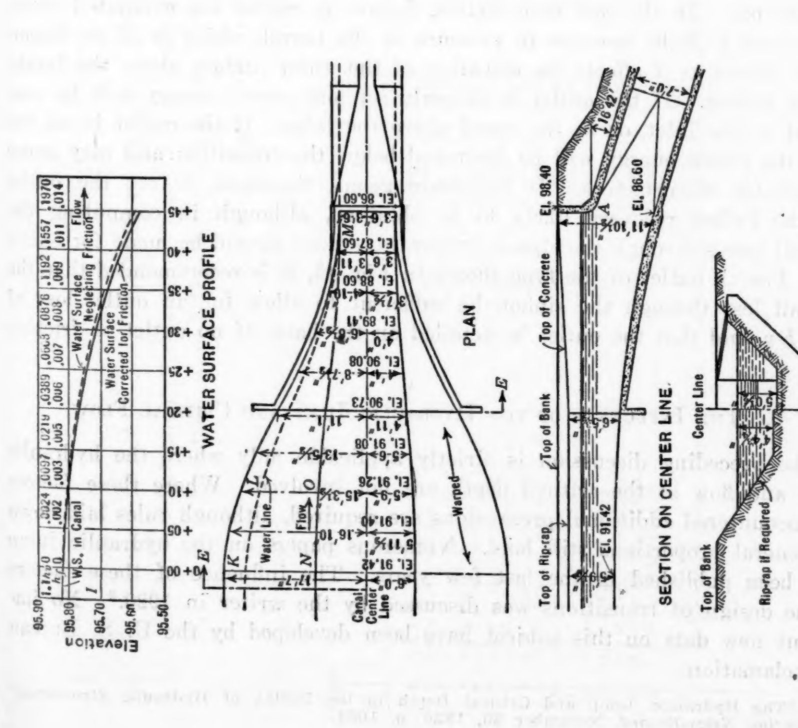


FIG. 14.—TYPICAL CONCRETE SIPHON INLET.

carried to the end of the transition, but the computed dimensions at Stations 0 + 40 and 0 + 45 are ignored. The conduit floor is drawn to a smooth connection between other computed points and the floor of the siphon barrel. With these exceptions the computations in Table 3 are similar to those in Table 1. The fact that the conduit is covered at points of high velocity makes it possible to take the liberties mentioned.

Example of a Siphon Outlet. — The necessity for holding the top of the siphon barrel down at the head-wall does not exist at a siphon outlet, and computed dimensions may be followed throughout. The same condition as to change of velocity under cover exists as at the inlet. Fig 15 represents a typical outlet design, the computations for which are shown in Table 4. The water-surface profile is taken as a single parabola, as recommended for a siphon inlet. As it is not desirable to curve the floor of the siphon barrel this procedure usually results in a vertical angle at the intersection of the siphon and the floor of the transition. An angle in the floor is less objectionable than an angle in the side walls, and, if small, apparently does little harm. For an outlet such as that shown in Fig. 15, the action can be improved by arbitrarily rounding the floor angle to a short radius.

Recovery conditions at a siphon outlet are not the same as at a flume outlet. If a flume outlet fails to recover the head demanded of it, the water is backed up in the flume and the free-board is decreased. It is desirable, therefore, to design for only a part of the probable recovery, as previously pointed out. In the case of a siphon, failure to realize the computed recovery means a slight increase in pressure in the barrel, which is of no importance, except as it affects the elevation of the water surface above the intake to the siphon. If the outlet is properly set any excess energy will be consumed at the inlet, or in the canal above the inlet. If the outlet is set too high the excess energy will be destroyed below the transition and may cause undesirable disturbances. It is advantageous, therefore, to set the outlet for the fullest recovery likely to be obtained, although in computing the over-all losses through the siphon proper, allowance should be made for outlet loss. For an outlet of the type shown in Fig. 15, it is recommended that the over-all loss through the siphon be sufficient to allow for an outlet loss of $0.2\, \Delta h v$, and that the outlet be detailed on the basis of no outlet or friction loss.

3.—The Influence of the Hydraulic Jump and Critical Flow

The preceding discussion is strictly applicable only where the hydraulic jump and flow at the critical depth are not involved. Where these factors are encountered additional precautions are required, although rules laid down for general proportions still hold. Numerous papers on the hydraulic jump have been published in the last few years. The influence of these factors on the design of transitions was discussed by the writer in 1920.* No important new data on this subject have been developed by the U. S. Bureau of Reclamation.

* "The Hydraulic Jump and Critical Depth in the Design of Hydraulic Structures," *Engineering News-Record,* November 25, 1920, p. 1034.

TABLE 4.—COMPUTATIONS FOR SIPHON OUTLET.

In siphon, $V = 3.93$ $h\,v = 0.240$ Elevation, water surface, Station $0 + 45 = 95.29$.
In canal, $V = 2.31$ $h\,v = 0.083$ Outlet loss neglected.

$\Delta\,h\,v = 0.157$ Water surface, single parabola.

Line.	Item.	STATION. 0 + 00	0 + 05	0 + 10	0 + 15	0 + 20	0 + 25	0 + 30	0 + 35	0 + 40	0 + 45
1	$\Delta\,h\,v = \Delta\,W.\,S.$ = Rise in $W.\,S.$*	......	0.033	0.062	0.087	0.109	0.126	0.140	0.149	0.155	0.157
2	$h\,v = 0.240 - \Delta\,h\,v$	0.240	0.207	0.178	0.153	0.131	0.114	0.100	0.091	0.085	......
3	V	3.93	3.65	3.38	3.14	2.90	2.71	2.54	2.42	2.34	......
4	Area = $Q \div V$	52.95	57.00	61.42	66.30	71.60	77.05	82.00	86.00	89.00	91.52
5	$0.5\,T$ = Half width at $W.\,S.$	3.500	4.100	4.970	6.110	7.500	9.250	11.050	12.730	14.030	14.800
6	$0.5\,B$ = Half bottom width	3.500	3.850	4.150	4.450	4.800	5.150	5.500	5.800	5.950	6.000
7	$0.5\,T + 0.5\,B$ = Ave. width	7.000	7.950	9.120	10.560	12.300	14.400	16.550	18.530	19.980	20.800
8	d = Area ÷ Ave. width	7.570	7.180	6.730	6.280	5.820	5.360	4.950	4.640	4.450	4.400
9	$W.\,S.$ Elev. = $95.133 + \Delta\,W.\,S.$	95.133	95.166	95.195	95.220	95.242	95.259	95.273	95.282	95.288	95.290
10	Grade = $W.\,S.$ Elev. $- d$	87.563	87.986	88.465	88.940	89.422	89.909	90.323	90.642	90.838	90.890
11	$0.5\,T - 0.5\,B$	......	0.250	0.820	1.660	2.700	4.100	5.550	6.930	8.080	8.800
12	Side slopes	......	0.348	0.122	0.264	0.463	0.766	1.122	1.493	1.815	2.000
13	H = Height of lining	10.337	9.694	8.995	8.300	7.598	6.891	6.257	5.938	5.742	5.690
14	$0.5\,W - 0.5\,B$ = Side slope × H	......	0.337	1.098	2.190	3.520	5.280	7.025	8.880	10.430	11.380
15	$0.5\,W$ = 0.5 Top width	3.500	4.187	5.248	6.640	8.320	10.430	12.525	14.680	16.380	17.380
16	$0.5\,W$ to nearest ⅛ in.	3 ft. 6 in.	4 ft. 2 in.	5 ft. 3 in.	6 ft. 7½ in.	8 ft. 4 in.	10 ft. 5 in.	12 ft. 6 in.	14 ft. 8 in.	16 ft. 4 in.	17 ft. 4⅛ in.

* Friction and outlet loss ignored in detailing outlet structure. In computing total drop through siphon allow for outlet loss $= 0.2\,\Delta\,h\,v + \Sigma\,h_f$.

It may be well to emphasize again the fact that the unexpected and unnecessary introduction of critical flow conditions is a frequent source of trouble in transitions. This condition is illustrated in the outlet shown in Fig. 5. Although the total length of this structure is 100 ft., the velocity actually increases to a point near the lower end, where it suddenly "jumps" to approximately canal velocity. The normal depth in both the tunnel and the canal is above critical, and if proper attention had been given to the detailed dimensions, the jump would have been avoided and the efficiency of the outlet increased. In addition, heavy wave action in the canal would have been avoided.

A situation of this kind is not likely to escape notice with the plan of computation recommended in this paper. However, it is essential that any approach to the critical depth be carefully noted. Before attempting to plan an important transition the designer should be thoroughly familiar with the phenomenon of critical flow.

Impact Troubles in Long Siphons.—As previously mentioned, the excess energy in a pipe line is consumed in the canal above the inlet, in the inlet, or in the upper end of the pipe. If the excess fall is small it is usually absorbed in a moderate increase in velocity in the canal, which does little harm. With greater fall, racing in the canal may become serious, and a check may be required at or near the intake to the siphon. The destruction of energy is thus transferred to the inlet.

In irrigation conduits, pipe lines are seldom controlled by gates or valves, the fall provided being only that required to maintain flow at maximum capacity. In the interest of conservatism the allowance for losses is usually greater than the actual requirement. If the pipe line is long the excess fall may be too great to be entirely absorbed in the inlet, and the water will race down the pipe for a distance at part depth, to be checked suddenly where it meets the back-water from the outlet. In large siphons of considerable length, operating at part capacity, this condition often results in serious vibrations. Because of certain manifestations this phenomenon is frequently referred to as "air trouble," and many unsuccessful attempts have been made to relieve the vibrations by the installations of vents, or by revision of the inlet structure. This is not a transition problem and is treated here only because it is often considered to be related to the inlet structure.

An example of a pipe line operating in this way is shown in Fig. 16. This siphon, which is on the King Hill Project, in Idaho, was constructed by a predecessor of the U. S. Bureau of Reclamation at a time when the project was expected to embrace a larger area than that ultimately developed. The capacity is considerably in excess of the actual requirements. The conditions shown in Fig. 16 were observed in July, 1925, with a flow of 120 sec-ft. With this discharge a heavy rumbling and a very noticeable vibration were noted some distance down the pipe from the inlet.

The disturbance appeared to lie wholly below the hydraulic gradient computed from the outlet structure as a control. This is in accordance with theory. It will be noted that the flow theoretically changes suddenly through

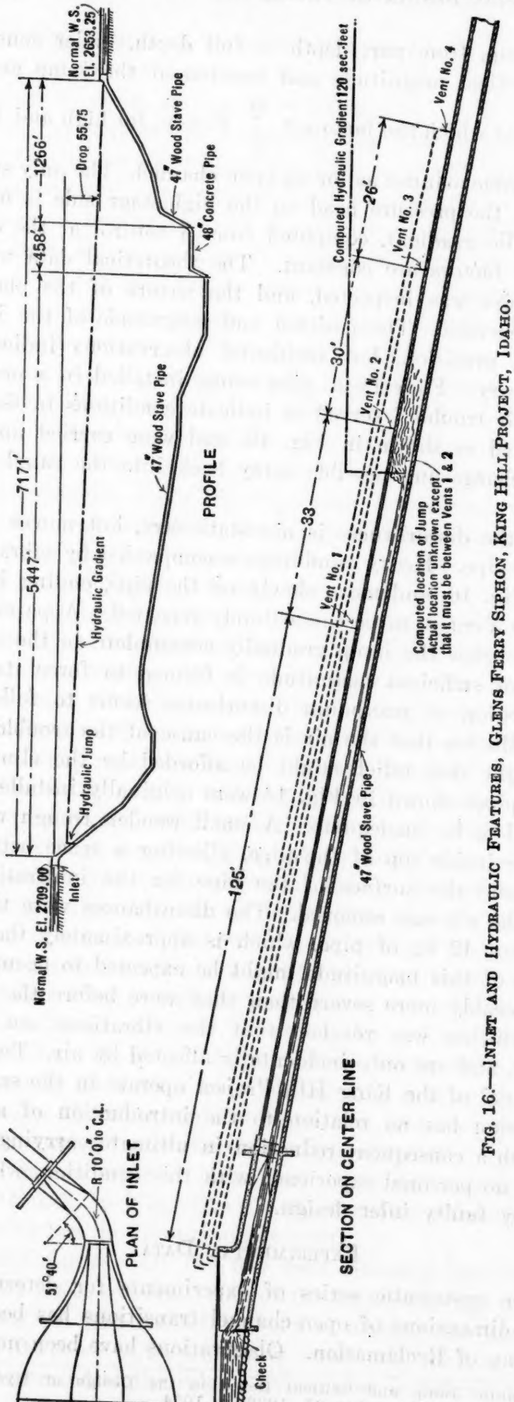


FIG. 16.—INLET AND HYDRAULIC FEATURES, GLENS FERRY SIPHON, KING HILL PROJECT, IDAHO.

the hydraulic jump, from part depth to full depth, under considerable pressure. The theoretical magnitude and location of the jump are obtained by finding the point at which the factors,* $\frac{Q}{g} V + p$, for high and low-stage flow, are equal, in the same manner as for an open channel. The only special features in a pipe is that the pressure head on the high-stage side is measured down from the hydraulic gradient, computed from a control at the outlet, and all other high stage factors are constant. The theoretical data were not available when the pipe was inspected, and the nature of the phenomenon was not fully comprehended. The position and magnitude of the jump were not determined with precision, but incidental observations indicate close conformity with theory. Four 1½-in. pipe vents, installed in a previous attempt to relieve the "air trouble," served to indicate conditions in the pipe. These vents were located as shown in Fig. 16, and were carried up the pipe and arranged to discharge any possible spray back into the canal over the inlet head-wall.

The seat of the disturbances is not stationary, but moves over a considerable length of pipe. Heavy rumblings accompanied by vibrations originate below Vent 3, Fig. 16, and move slowly up the pipe, ending in an explosion at a point above Vent 2, to be immediately repeated. Apparently, small air-bubbles released below the jump gradually accumulate in the top of the pipe until a pocket of sufficient magnitude is formed to force its way back up stream. The region of maximum disturbance seems to follow the bubble, producing the illusion that the air is the cause of the trouble.

It was thought that relief might be afforded by the elimination of the air. The vent pipes shown in Fig. 16 were originally installed for this purpose, but proved to be inadequate. A small wooden trough was temporarily bolted along the inside top of the pipe, allowing a space between the edges of the trough and the surface of the pipe for the infiltration of air, and apparently all the air was removed. The disturbances were then confined to a length of about 12 ft. of pipe, which is approximately the length that a hydraulic jump of this magnitude might be expected to occupy. The vibrations were noticeably more severe than they were before the removal of the air. The conclusion was reached that the vibrations are caused by the hydraulic jump, and are only incidentally affected by air. Two other siphons on the main canal of the King Hill Project operate in the same way.

This discussion has no relation to the introduction of air into siphons at full flow with a consequent reduction in ultimate carrying capacity. The writer has had no personal experience with this condition, which is supposed to be caused by faulty inlet design.

Experimental Data

No complete systematic series of experiments for determining the correct form and dimensions of open-channel transitions has been attempted by the U. S. Bureau of Reclamation. Observations have been made on a limited

* See "Hydraulic Jump and Critical Depth in the Design of Hydraulic Structures," *Engineering News-Record*, November 25, 1920, p. 1034.

number of existing structures, some of which have been mentioned previously. The results of others are shown on Fig. 17. These tests consist principally of measurements of outlet and entrance losses.

Methods and Precision.—The approximate nature and inconsistency of some of the data are readily apparent. For example, the abrupt inlet (*a*), Fig. 17, shows practically no entrance loss, whereas the comparatively slender inlet, (*b*), shows a loss of 46% of the change in velocity head. Practically all the head losses shown on Fig. 17 were obtained from level readings with the end of a level rod held on the water surface. There is always sufficient oscillation to destroy the precision of readings taken in this way, especially where the fall to be measured is small. Even with comparatively smooth flow, maximum fluctuations of as much as an inch may occur. An error of ½ in. would materially affect many of the coefficients shown on Fig. 17.

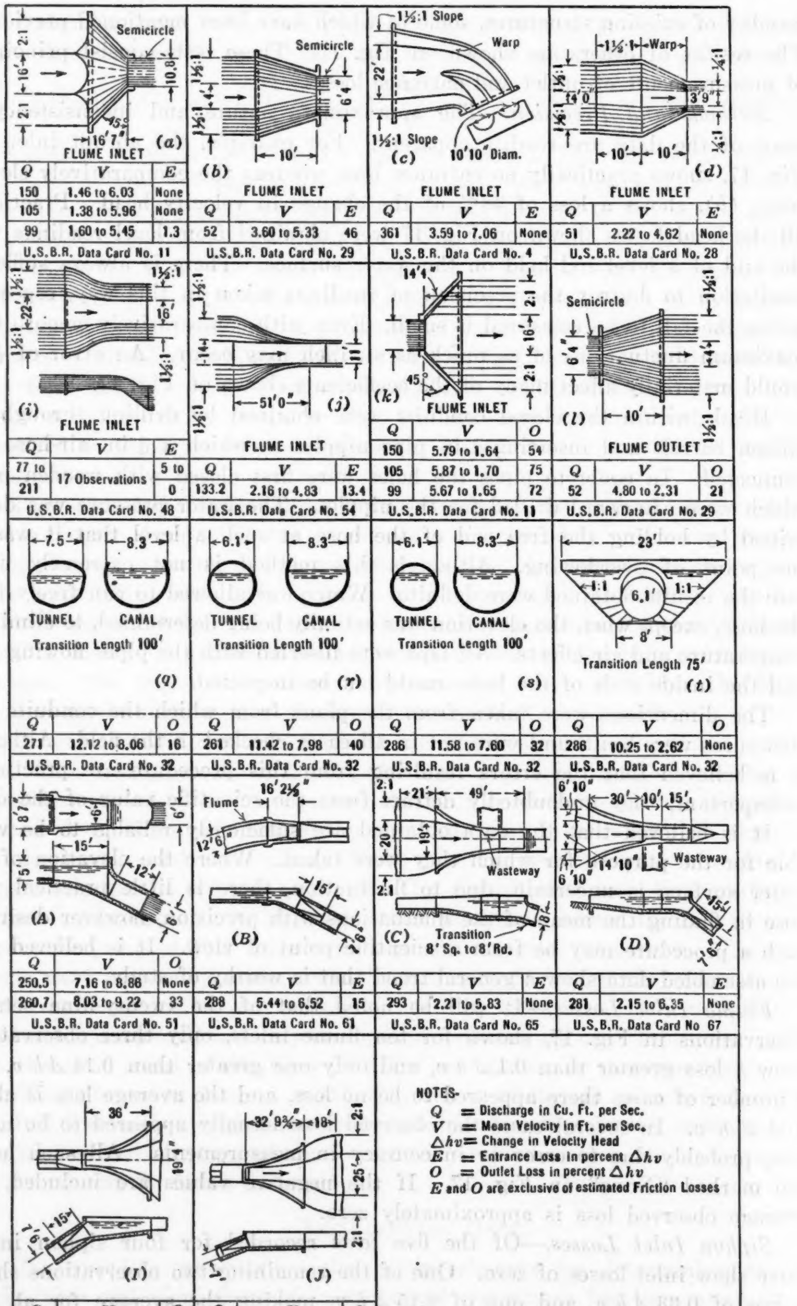
Heads within the closed conduits were obtained by drilling through the siphon barrels and inserting ⅛-in pipe nipples to which a ¼-in. air-hose was connected. In concrete pipes the holes were first closed with wooden plugs which were afterward drilled for the nipples. The water pressure was determined by holding the free end of the hose at such a level that it was on the point of overflowing. Although this method is not extremely accurate the results obtained were definite. Water was allowed to run freely from the hose, except when the elevation was actually being determined, to eliminate temperature and air effects. All taps were inserted with the pipes flowing full, and the inside ends of the holes could not be inspected.

The dimensions were taken from the plans from which the conduits and structures were built, and were not in all cases checked in the field. Although it is believed that the errors resulting from this procedure are practically unimportant, they undoubtedly detract from the scientific value of the data.

It is believed that the data obtained are sufficiently reliable to be valuable for the purpose for which they were taken. Where the elevation of the water surface is uncertain, due to fluctuation, there is little practical purpose in finding the mean of the fluctuations with precision, however desirable such a procedure may be from a scientific point of view. It is believed that the assembled data show a general trend that is worthy of study.

Flume Inlet Losses.—It will be noted that of the twenty-nine sets of observations in Fig. 17, shown for ten flume inlets, only three observations show a loss greater than $0.1\,\Delta h v$, and only one greater than $0.14\,\Delta h v$. In a number of cases there appeared to be no loss, and the average loss is about $0.04\,\Delta h v$. In several cases the observed loss actually appeared to be negative, probably due to error or inaccuracy in measurements. All such losses are marked "None" on Fig. 17. If the negative values are included, the average observed loss is approximately zero.

Siphon Inlet Losses.—Of the five tests recorded for four siphon inlets, three show inlet losses of zero. One of the remaining two observations shows a loss of $0.33\,\Delta h v$, and one of $0.15\,\Delta h v$, making the average for all five approximately $0.1\,\Delta h v$. As previously explained, all the excess head in a siphon is concentrated at the inlet. This makes it difficult to secure a rating



(a) FLUME INLET

Q	V	E
150	1.46 to 6.03	None
105	1.38 to 5.96	None
99	1.60 to 5.45	1.3

U.S.B.R. Data Card No. 11

(b) FLUME INLET

Q	V	E
52	3.60 to 5.33	46

U.S.B.R. Data Card No. 29

(c) FLUME INLET

Q	V	E
361	3.59 to 7.06	None

U.S.B.R. Data Card No. 4

(d) FLUME INLET

Q	V	E
51	2.22 to 4.62	None

U.S.B.R. Data Card No. 28

(i) FLUME INLET

Q	V	E
77 to 211	17 Observations	5 to 0

U.S.B.R. Data Card No. 48

(j) FLUME INLET

Q	V	E
133.2	2.16 to 4.83	13.4

U.S.B.R. Data Card No. 54

(k) FLUME INLET

Q	V	O
150	5.79 to 1.64	54
105	5.87 to 1.70	75
99	5.67 to 1.65	72

U.S.B.R. Data Card No. 11

(l) FLUME OUTLET

Q	V	O
52	4.80 to 2.31	21

U.S.B.R. Data Card No. 29

(q)

Q	V	O
271	12.12 to 8.06	16

U.S.B.R. Data Card No. 32

(r)

Q	V	O
261	11.42 to 7.98	40

U.S.B.R. Data Card No. 32

(s)

Q	V	O
286	11.58 to 7.60	32

U.S.B.R. Data Card No. 32

(t)

Q	V	O
286	10.25 to 2.62	None

U.S.B.R. Data Card No. 32

(A)

Q	V	O
250.5	7.16 to 8.86	None
260.7	8.03 to 9.22	33

U.S.B.R. Data Card No. 51

(B)

Q	V	E
288	5.44 to 6.52	15

U.S.B.R. Data Card No. 61

(C)

Q	V	E
293	2.21 to 5.83	None

U.S.B.R. Data Card No. 65

(D)

Q	V	E
281	2.15 to 6.35	None

U.S.B.R. Data Card No 67

(I)

Q	V	O
281	6.36 to 1.94	2.5

U.S.B.R. Data Card No. 68

(J)

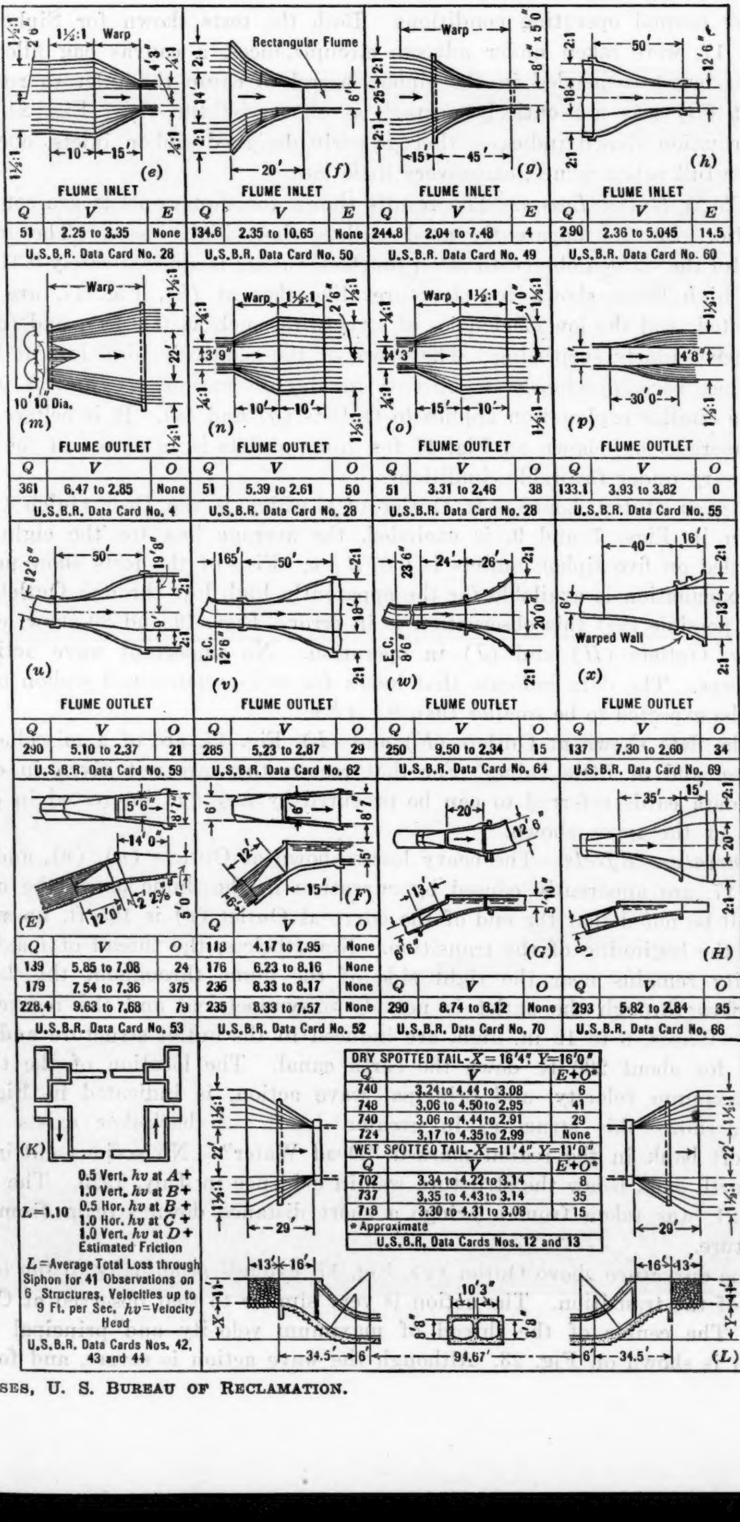
Q	V	O
252	6.55 to 1.68	17

U.S.B.R. Data Card No. 63

NOTES-

Q = Discharge in Cu. Ft. per Sec.
V = Mean Velocity in Ft. per Sec.
Δhv = Change in Velocity Head
E = Entrance Loss in percent Δhv
O = Outlet Loss in percent Δhv
E and O are exclusive of estimated Friction Losses

FIG. 17.—TABULATED DATA ON TRANSITION



FLUME INLET

Q	V	E	Q	V	E	Q	V	E	Q	V	E
51	2.25 to 3.35	None	134.6	2.35 to 10.65	None	244.8	2.04 to 7.48	4	290	2.36 to 5.045	14.5
U.S.B.R. Data Card No. 28			U.S.B.R. Data Card No. 50			U.S.B.R. Data Card No. 49			U.S.B.R. Data Card No. 60		

FLUME OUTLET

Q	V	O	Q	V	O	Q	V	O	Q	V	O
361	6.47 to 2.85	None	51	5.39 to 2.61	50	51	3.31 to 2.46	38	133.1	3.93 to 3.82	0
U.S.B.R. Data Card No. 4			U.S.B.R. Data Card No. 28			U.S.B.R. Data Card No. 28			U.S.B.R. Data Card No. 55		

FLUME OUTLET

Q	V	O	Q	V	O	Q	V	O	Q	V	O
290	5.10 to 2.37	21	285	5.23 to 2.87	29	250	9.50 to 2.34	15	137	7.30 to 2.60	34
U.S.B.R. Data Card No. 59			U.S.B.R. Data Card No. 62			U.S.B.R. Data Card No. 64			U.S.B.R. Data Card No. 69		

Q	V	O	Q	V	O	Q	V	O	Q	V	O
			118	4.17 to 7.95	None						
139	5.85 to 7.08	24	176	6.23 to 8.16	None						
179	7.54 to 7.36	375	236	8.33 to 8.17	None						
228.4	9.63 to 7.68	79	235	8.33 to 7.57	None	290	8.74 to 6.12	None	293	5.82 to 2.84	35
U.S.B.R. Data Card No. 53			U.S.B.R. Data Card No. 52			U.S.B.R. Data Card No. 70			U.S.B.R. Data Card No. 66		

$L = 1.10$:
0.5 Vert. hv at A +
1.0 Vert. hv at B +
0.5 Hor. hv at B +
1.0 Hor. hv at C +
1.0 Vert. hv at D +
Estimated Friction

L = Average Total Loss through Siphon for 41 Observations on 6 Structures, Velocities up to 9 Ft. per Sec. hv = Velocity Head

U.S.B.R. Data Cards Nos. 42, 43 and 44

Q	V	E+O*
DRY SPOTTED TAIL-$X = 16'4''$, $Y = 16'0''$		
740	3.24 to 4.44 to 3.08	6
748	3.06 to 4.50 to 2.95	41
740	3.06 to 4.44 to 2.91	29
724	3.17 to 4.35 to 2.99	None
WET SPOTTED TAIL-$X = 11'4''$, $Y = 11'0''$		
702	3.34 to 4.22 to 3.14	8
737	3.35 to 4.43 to 3.14	35
718	3.30 to 4.31 to 3.08	15

* Approximate

U.S.B.R. Data Cards Nos. 12 and 13

LOSSES, U. S. BUREAU OF RECLAMATION.

under normal operating conditions. Both the tests shown for Siphon (*A*) Fig. 17, were taken under adverse circumstances, a canvas bag filled with water being suspended in the siphon barrel to cause the inlet to run full. Inlet (*B*) was not entirely normal, as shown by the view, Fig. 21. The information shown indicates that properly designed siphon inlets, operating under full submergence, cause very little loss.

Flume Outlet Losses.—The results shown for flume outlets are somewhat erratic. Several apparently good outlets show high losses. The average loss for the sixteen observations on fourteen outlets is approximately 0.31 $\Delta h v$. The high losses shown for structures like that at (*k*), Fig. 17, are to be expected, and the low efficiencies of structures, such as (*l*), (*n*), and (*o*), are not particularly surprising. The cause of the relatively high loss at Outlet (*r*) has been previously mentioned in the discussion of Figs. 5 and 8, and a similar explanation applies to Outlets (*q*) and (*s*). It is believed that the average loss shown on Fig. 17 for flume outlets is in excess of the losses necessary under favorable conditions.

Siphon Outlet Losses.—If Outlet (*E*), which is the Deer Gulch Outlet, shown in Figs. 7 and 9, is excluded, the average loss for the eight tests recorded on five siphon outlets is 0.07 $\Delta h v$. Five of the tests show no loss. No explanation is available for the apparently high loss through Outlet (*H*). It is possible that this observation is in error. Figs. 19 and 20 show, respectively, Outlets (*H*) and (*J*) in operation. No important wave action is apparent. The data indicate that losses for well-proportioned siphon outlets may be expected to be smaller than 0.1 $\Delta h v$.

The data shown at Outlets (*K*) and (*L*), Fig. 17, are of a miscellaneous nature, and of little value. For that reason they are not given in detail. The data cards referred to can be procured by any one interested in structures of the types shown.

Curvature Effects.—The heavy losses shown at Outlets (*u*), (*v*), and (*x*), Fig. 17, are apparently caused by curvature in the flume above the outlet. It will be noted that the end of the curve at Outlet (*v*) is 165 ft. up stream from the beginning of the transition. Nevertheless, the thread of maximum velocity remains near the right side of the flume throughout this length. The flow through the outlet is notably unsymmetrical and the recovery is poor. Waves, 6 to 12 in. high, are induced in the outlet structure and continue for about 200 ft. down the earth canal. The location of the thread of maximum velocity, and principal wave action, is indicated in Fig. 22, which shows this structure in greater detail. A back-flow exists along the left bank in the region marked "Dead Water". No serious cutting of the earth bank below the structure was in evidence in July, 1925. The view, Fig. 21, was taken from a bridge a short distance down stream from this structure.

The curvature above Outlet (*x*), Fig. 17, extends practically to the beginning of the transition. The action is very similar to that observed at Outlet (*v*). The center of the thread of maximum velocity and principal wave action is shown on Fig. 23. Although the wave action is severe, and follows

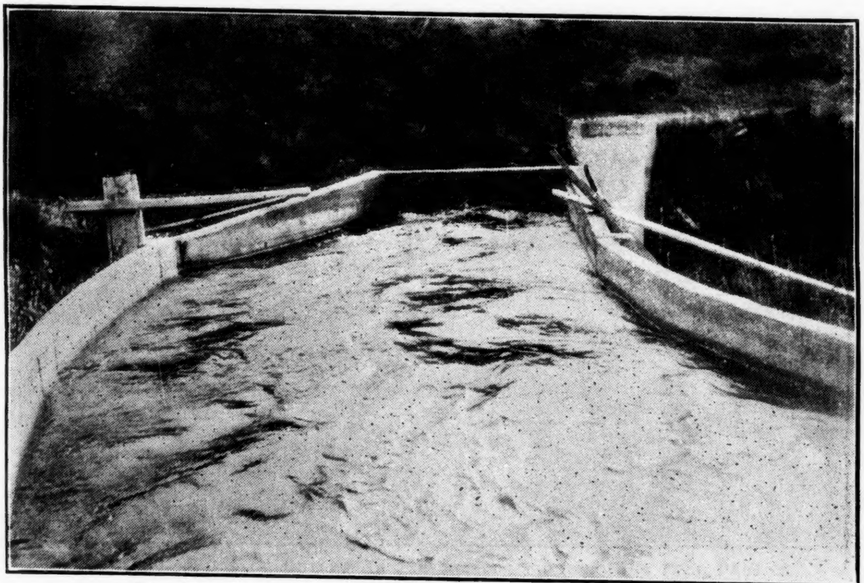


FIG. 18.—INLET, LITTLE CONCRETE SIPHON, KING HILL PROJECT.

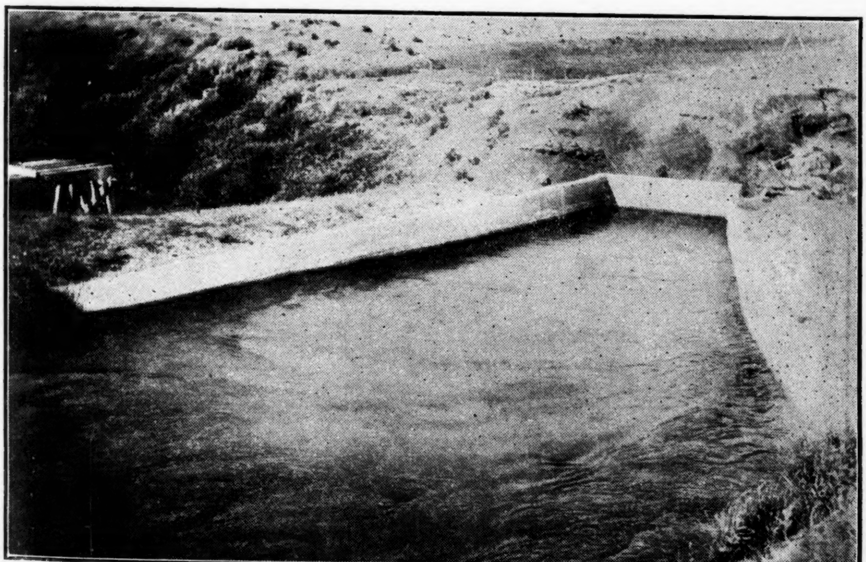


FIG. 19.—VIEW OF OUTLET, TOANA SIPHON, KING HILL PROJECT, $Q = 293$ SECOND-FEET.

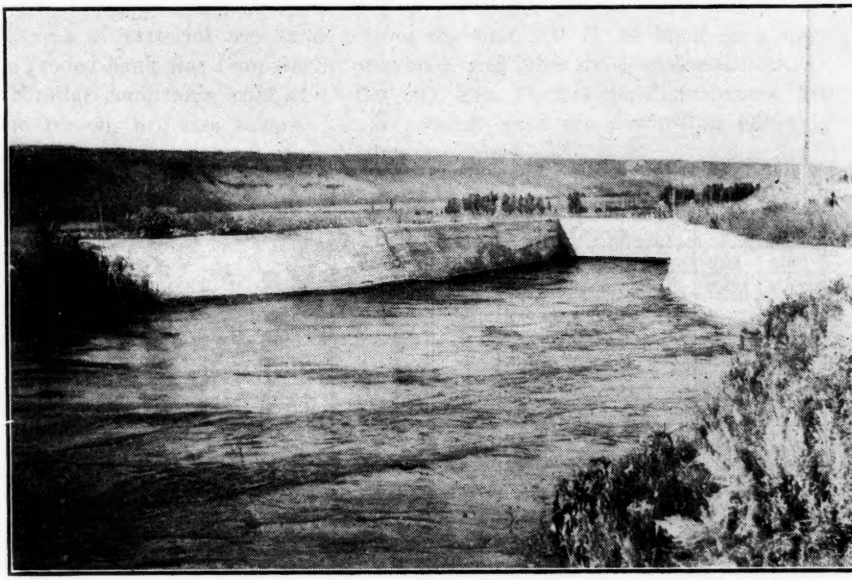


FIG. 20.—VIEW OF OUTLET, LITTLE PILGRIM SIPHON, KING HILL, PROJECT, $Q = 252$ SECOND-FEET.

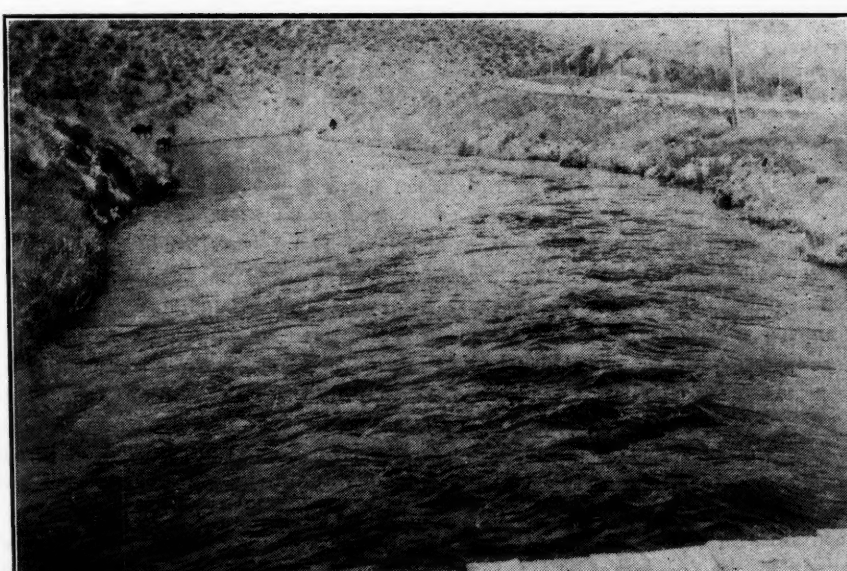
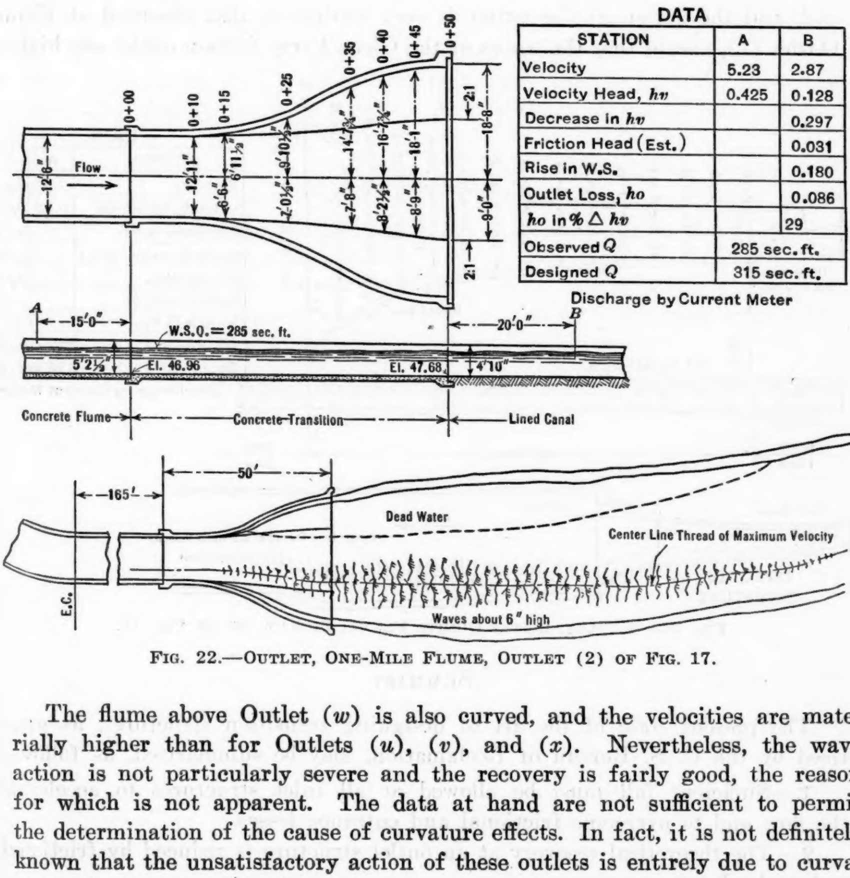


FIG. 21.—OUTLET, ONE-MILE FLUME, KING HILL PROJECT, SHOWING WAVE ACTION DUE TO CURVATURE IN FLUME ABOVE OUTLET.

the right bank of the earth canal closely, this structure operated several seasons with no apparent tendency to cut. Then cutting began suddenly and a 4-ft. thickness of material was eroded from the first 100 ft. of bank in a day. The eroded bank was temporarily protected, and later lined with concrete.

Similar conditions exist at Outlet (*u*), Fig. 17, but the disturbances, for some reason, are less severe. It is possible that the disturbing effect is partly offset by a reversal of curvature.



DATA

STATION	A	B
Velocity	5.23	2.87
Velocity Head, *hv*	0.425	0.128
Decrease in *hv*		0.297
Friction Head (Est.)		0.031
Rise in W.S.		0.180
Outlet Loss, *ho*		0.086
ho in % Δ *hv*		29
Observed Q	285 sec. ft.	
Designed Q	315 sec. ft.	

Discharge by Current Meter

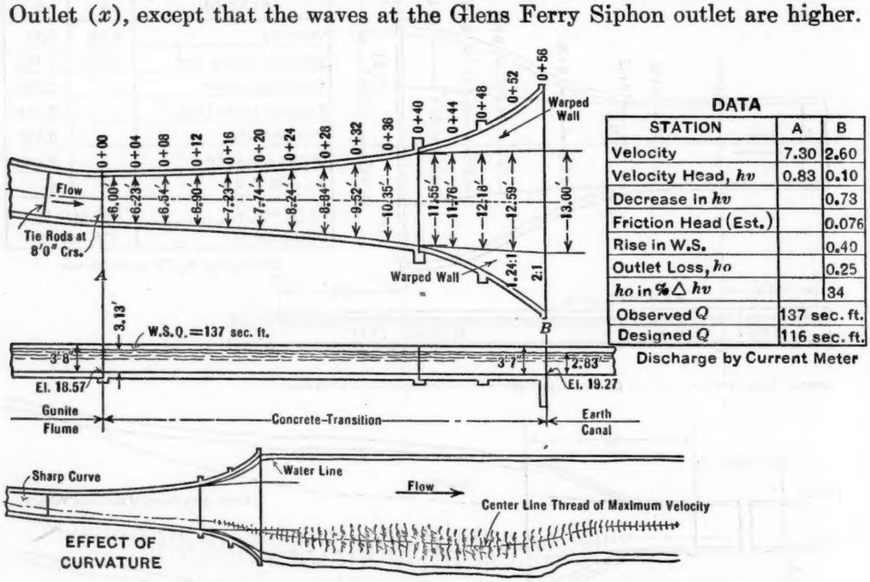
FIG. 22.—OUTLET, ONE-MILE FLUME, OUTLET (2) OF FIG. 17.

The flume above Outlet (*w*) is also curved, and the velocities are materially higher than for Outlets (*u*), (*v*), and (*x*). Nevertheless, the wave action is not particularly severe and the recovery is fairly good, the reason for which is not apparent. The data at hand are not sufficient to permit the determination of the cause of curvature effects. In fact, it is not definitely known that the unsatisfactory action of these outlets is entirely due to curvature. It is understood that other observers have noticed a tendency for the line of high velocity to be deflected to one side or the other at outlets from straight channels, the direction of deflection being apparently accidental.

Few comparable observations on outlets from flumes with and without curvature are available. The flume above Outlet (*p*), Fig. 17, is straight, but the velocities are low. Outlets (*k*), (*l*), (*m*), (*n*), and (*o*) are of such different forms as to render comparisons useless. Outlets (*q*), (*r*), and (*s*), as previously mentioned, involve the hydraulic jump. Outlet (*t*) appears to substantiate the assumption that the current deflection is due to curvature.

The tunnel above this structure is straight, and the transition appears to work efficiently, with moderately high velocities.

The action of the siphon outlets shown on Fig. 17 tends to disprove the theory of instability of outlet flow. All the pipes leading up to these outlets are straight in plan, and in no case was there evidence of unsymmetrical flow. However, the Glens Ferry Siphon, a profile of which is shown on Fig. 16, has a horizontal curve a short distance from its down-stream end, and the action at the outlet is very similar to that observed at Flume Outlet (x), except that the waves at the Glens Ferry Siphon outlet are higher.



DATA

STATION	A	B
Velocity	7.30	2.60
Velocity Head, hv	0.83	0.10
Decrease in hv		0.73
Friction Head (Est.)		0.076
Rise in W.S.		0.40
Outlet Loss, ho		0.25
ho in % Δ hv		34
Observed Q	137 sec. ft.	
Designed Q	116 sec. ft.	

Discharge by Current Meter

FIG. 23.—OUTLET, LITTLE GUNITE FLUME, OUTLET (x) OF FIG. 17.

SUMMARY

The present state of the art of designing transition structures, as practiced by the U. S. Bureau of Reclamation, may be summarized, as follows:

1.—Sufficient fall must be allowed at all inlet structures to accelerate the flow and to overcome frictional and entrance losses.

2.—The theoretical recovery at an outlet structure is reduced by frictional and outlet losses.

3.—At open channel outlets a small factor of safety may be obtained by setting the transition for less than its maximum recovering capacity, but erosion below the structure may be slightly increased.

4.—At siphon outlets a small factor of safety may be obtained and erosion avoided by setting the transition for more than its assumed recovering capacity.

5.—Simple designs may be prepared by adapting the details of previous designs, known to be satisfactory, if proper allowance is made for loss of head.

6.—Important structures, where velocities are high, must be carefully designed to conform to a smooth theoretical water surface. Sharp angles must be avoided.

7.—Horizontal curvature in the conduit above an outlet appears to reduce its efficiency, and to produce objectionable cutting velocities in the canal below.

8.—The transition loss through an inlet of the type shown in Fig. 12 or Fig. 14, is likely to be less than $0.05\,\Delta h\,v$. An allowance of $0.1\,\Delta h\,v$ is safe for use in designing.

9.—The outlet loss at a properly designed transition of the type shown in Figs. 13 and 15, may be expected to be less than $0.1\,\Delta h\,v$, unless the conduit above the structure is curved. An allowance of $0.2\,\Delta h\,v$ is safe for use in designing.

10.—No definite data as to the best form of water surface profile, best form of structure, or most efficient length of transition, are available.

11.—Special care is required where the critical depth is approached or where the hydraulic jump is involved.

12.—The disturbances often observed in long, uncontrolled siphons, at part capacity, are not caused by entrained air, but by the hydraulic jump in the pipe.

us
log
so
ne

be
on
Se
dif

tin
sin
no
als
to
hav
pro
for

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

WATER-POWER APPRAISALS

By William H. Cushman,* M. Am. Soc. C. E.

Synopsis

This paper calls attention to the lack of uniformity in the methods now used in appraising water-power property. An attempt is made to outline a logical method of procedure that will eliminate several factors which, under some of the current methods, are difficult to determine, or which lead to erroneous conclusions.

Three methods are commonly used in approaching this problem, namely:

1.—The Steam Substitution Method;
2.—The Real Estate Sales Method; and,
3.—The Capitalized Net Earnings Method.

1.—Steam Substitution Method

It would appear, offhand, that the value of a block of water power could be determined by finding the value of an equivalent quantity of steam power, on the principle that "things equal to the same thing are equal to each other". Second thought exposes the fallacy of this reasoning, as the two blocks of differently produced power are not equal, and cannot be made equal.

A block of steam power can be produced from a given power plant continuously, at a cost capable of being definitely and closely estimated. A similar block of water power, produced from a plant developed to the economic limit, involves, not only an installation of hydraulic machinery, but also, in most instances, an auxiliary source of power to supply deficiencies due to low-water periods. This may mean that a secondary steam plant would have to be maintained. The capacity of such an auxiliary may be a large proportion of that of the hydraulic installation and may be required only for short and intermittent periods aggregating, say, about 25% of the year.

Note.—Written discussion on this paper will be closed in February, 1928.
* Cons. Engr. (Hydr.), Watertown, N. Y.

To produce a given amount of power by water ordinarily means duplicate installations of power-producing machinery and, consequently, a greater installation cost per horse-power. The flexibility of a steam power plant enables the user to adapt (within close limits) the power produced to that required from time to time. Ordinarily, water power has to be taken as it comes, even if storage reservoirs partly equalize the stream flow.

The Courts almost invariably resist attempts to value a water power by the steam substitution method, and the underlying reasons for this are undoubtedly based, in whole or in part, on the inequalities of plant installation cost and on the flexibility of power supply. A method of valuation that has no standing in Court is unavailable for an appraisal engineer.

2.—Real Estate Sales Method

The Real Estate Sales Method is based on current sales of water-power property. At first sight, this also seems to have all the weight of "business judgment of values" behind it, or of a so-called "practical" valuation.

This method is always accorded full attention by the Court in water-power valuation cases, but undoubtedly considerable of its standing is due to its being stated in definite figures that do not involve the study of data, often of a highly technical character, by the Court.

In 1909, the New England Water Works Association published a report* that contains the details of numerous sales of water power, the dates of the sales ranging from 1878 to 1906. A study of these recorded sales shows a range between \$84.70 and \$1 470.00 per h.p. It is manifestly impossible to reconcile such wide differences.

The reasons for these differences tend to show the unreliability of the Real Estate Sales Method. These basic reasons are that sales of water rights are relatively infrequent, so that there is no established market price for such property. Also, practically all water-power rights, within transmission distance of a market for the power, have long since been acquired and are closely held, the only transfers being "forced sales" and sales of fractional portions of a general development, these fractional portions showing values disproportionate to the value of the development as a whole—that is, "hold-up" prices on the one hand as against nominal prices for portions having no heads worth developing by themselves. In the absence of an active market, dealing with water-power real estate, casual sales do not reflect reliable or consistent data on values.

3.—Capitalized Net Earnings Method

Capitalized net earnings would seem to be a more indirect method of valuation than either of the preceding methods described, but, if the factors used are correctly employed, this method will produce logical and consistent values. Water power produced in the presence of an ample market has a value as fixed and stable as the values of most other articles of commerce.

---

* Report of Committee Appointed to Collect Data Relating to Awards for Water and Water Power Diversions, *Journal*, New England Water Works Assoc., Vol. XXIV, March, 1910, No. 1, p. 1.

If a horse-power can be readily sold to one of several industries, its value is stabilized and established within very narrow limits.

Under this method the procedure is to consider the power as sold at current market rates, thus producing an annual gross income. The values of the water-power rights are determined by estimating the cost of an entirely new plant, of a capacity equal to the economic limit of development. Then, the annual cost of operating this plant must be estimated.

From the annual gross income the annual operating cost is deducted to show the annual net earnings which, capitalized at a proper rate, give the value of the water power developed to the "economic limit".

If the estimated cost of the "entirely new plant", which constitutes the "development", be deducted from the value thus arrived at, the remainder must represent the value of the undeveloped water rights.

Capitalized Savings

A variation of the Capitalized Net Earnings Method has been used to capitalize the savings. With the principles strictly adhered to, this method might work out correctly, but the pathway is strewn with pitfalls. The assumption on the "earnings" basis is that the owner of the power can sell it at wholesale rates to, say, a public service corporation operating in his district, or to other parties who will pay as much or more for it, this being common practice.

The assumption on the "savings" basis is that the owner uses all his power and saves the difference between his cost of production and what a like amount of power purchased from outside sources, but not necessarily at wholesale rates, would cost him.

Practically no owner uses all his power to the "economic limit of development", therefore, he would not make the calculated "saving", in actual practice, as substituted power would be required in lesser amounts to fulfill his power requirements, and he would "save" only on the basis of this lesser amount.

It would be impossible for him to use such of his power as his business required and sell the surplus at regular, wholesale rates, such as he would receive if he sold the whole block of power; the value of his surplus would vary over a wide range according to its volume and hours of availability. Therefore, in the ordinary case, a calculated "saving" would be a highly theoretical quantity.

In the numerous instances with which the "savings" method has been used, almost invariably error has crept in, due usually to calculating the "saving" to the particular industry in which the power happened to be used. This undoubtedly gave the true value to the user, but might be far afield as to the value that could have been realized from selling the block of power in the open market. It seems much safer to consider the power as being sold at the current market rates.

Existing Development

In appraising the value of any particular water power as of a given date, it is necessary to segregate and value physical development, such as structures and machinery, then depreciate it according to age and condition to give it a "sound" value that may be added to the value of the water rights, as separately determined.

Land

Land sufficient to hold the riparian rights and permit of their development is part and parcel of a water-power property. As such, therefore, it should not be given any separate value.

Surplus land, not so required, should be appraised under its proper classification as another asset.

Hydraulic Data

The essential data for determining the quantity of power, are net head, turbine efficiency, and available stream flow.

Available stream flow is considered to be limited to the highest flow that can be profitably developed. In special cases this might mean that the highest average monthly flow could be utilized; but, generally speaking, it is not profitable to develop for a capacity in excess of the flow shown to be available during the eighth, or ninth, dryest month.

In assembling the monthly average flows, during a cycle of years of gaugings, it has become current practice to average the months in the order of their dryness rather than by calendar months. There does not seem to be any good reason for this method as, if it is carried to its logical conclusion, a 12-year cycle might show a different month each year as the lowest month.

It is difficult to conceive what useful purpose an average of the dryest months would serve. What is essential to power operation is a knowledge of how much water, on the average, is available each calendar month in the year.

After assembling the figures for the average dryest month, made up of January, June, etc., this value will not be much lower, as a rule, than by the calendar-month method. On the other hand, the ninth dryest month will show a correspondingly higher average, so that the horse-power years might well be the same quantity using either method and only the quantities of primary and second-class power be changed.

Experienced plant operators will agree that, within any month, fluctuations may occur having a duration ranging from a few hours to several days and that these fluctuations establish the minimum peak.

Horse Power

Table 1 shows the process of determining the available horse-power years as clearly as is necessary, but the main object is the selection of a definite example to be carried through to a valuation. This table is taken from an

appraisal report verbatim. The head used is 31 ft., the drainage area, 1 890 sq. miles, and the monthly average flows are averages of all Januarys, Februarys, etc., over 28 years of gaugings. The economic limit of development in this instance is taken to be 3 000 sec-ft. and all additional flow is disregarded as unavailable.

TABLE 1.—EXAMPLE OF AVAILABLE PRIME AND SECOND-CLASS HORSE POWER.

Month.	Second-feet per square mile.	Total flow, in second-feet.	Available flow, in second-feet.	Flow, including 300 sec-ft. of storage.	Horse-power at 85% efficiency; 31-ft. head.	Primary horse power.	Second-class horse power.
January........	1.754	3 315	3 000	.....	8 983	5 267	3 716
February.......	1.609	3 041	3 000	.....	8 983	5 267	3 716
March..........	3.216	6 078	3 000	.....	8 983	5 267	3 716
April..........	5.469	10 336	3 000	.....	8 983	5 267	3 716
May............	2.717	5 135	3 000	.....	8 983	5 267	3 716
June...........	1.434	2 710	2 710	.....	8 115	5 267	2 848
July...........	0.894	1 690	1 690	1 990	5 959	5 267	692
August.........	0.772	1 459	1 459	1 759	5 267	5 267	.....
September	0.795	1 503	1 503	1 803	5 399	5 267	132
October........	1.326	2 506	2 506	.....	7 504	5 267	2 237
November.......	1.680	3 175	3 000	.....	8 983	5 267	3 716
December	1.870	3 534	3 000	.....	8 983	5 267	3 716
Horse-power years.........	.....	.....	.....	.....	.....	5 267	2 660

Stored water, to the extent of 300 sec-ft., is released during the three months' period of low flow; 85% is used as the average turbine efficiency.

LOAD FACTOR

It is impossible, of course, for any water-power owner to use all the available power all the time; allowing for a reduced use on Sundays and holidays, possibly 85% of the power at the turbine shaft could be considered as the horse power actually available. If, however, the power is to be converted into electrical power, for supplying the wholesale market, 75% would more nearly represent the available power supply.

MARKET VALUE

The best evidence of the market value of power is the price shown by existing contracts, for like quantities of power, in that particular locality. Existing contracts require careful study to develop the actual price paid per horse-power-year. Varying peak-load methods of measurement, different power factors, and "demand-plus-energy-charge" contracts, have to be taken into account.

In some fields so-called "super-power" groups, consisting of otherwise independent public service corporations, exchange power at prices that possibly represent the minimum wholesale rates better than the ordinary sliding scale forms of contract. This exchange is governed by a different method

of classification than that heretofore used. Primary power is all power sold during 10 daylight hours; second-class power is all power sold during the other 14 hours of the day.

Given these two rates, it is a simple calculation to determine the wholesale value of the two classes of power. The outstanding feature of this method of exchange is the economic results obtained by reducing, or eliminating, steam auxiliary requirements.

Power Prices

For the purpose of carrying through a definite valuation, the arbitrary assumption of \$42 and \$20 per horse-power-year, for primary and second-class power, respectively, is used in the following illustration of the method recommended for consideration.

At these rates the amounts of power shown in Table 1 would have the following values:

5 267 h.p.-years at 75% factor	= 3 950	@ \$42	\$165 900
2 660 " " " "	= 1 995	@ 20	39 900
Gross annual income			\$205 800

Cost of Development

It may be noted from Table 1 that in order to produce 5 945 h.p.-years, the development to the maximum economic limit of 3 000 sec-ft. is required, and that this is equivalent to 8 983 h.p., or, practically, a 9 000-h.p. plant.

Again, for the purpose of illustration, the estimated cost of a new hydro-electric plant of 9 000-h.p. capacity is given, as follows:

Excavation, 10 000 cu. yd. at \$4	\$40 000
Concrete construction, 18 000 cu. yd. at \$20	360 000
Power-house superstructure	36 000
Hydraulic turbines, 9 000 h.p. at \$12	108 000
Generators, switchboard, etc., 7 500 kv-a. at \$20	150 000
Crane, trash racks, and auxiliary items	20 000
Total	\$714 000
Contingencies, 10%	71 400
Total	\$785 400
Engineering design and supervision, 6%	47 124
Organization, legal and miscellaneous expenses, 4%	31 416
Cost of development	\$863 940

Annual Operating Cost

The estimated cost per annum of operating a 9 000-h.p. hydro-electric plant, as hereinafter used, is exclusive of carrying charges, such as interest on investment and sinking fund, which have to do with financing rather than operating the plant.

This computation is being made to show the interest-paying capacity of the fully developed power. If the interest and sinking fund charges were deducted from the earnings, the value of the plant ($863 940) would not be deducted, as under that method of retiring bonds at maturity, the financing of the development would have been provided for in perpetuity.

In financing a power project it is customary to issue bonds, preferred stock, and common stock. The bonds would bear a certain rate of interest and their value would vary according to the market for such securities. Preferred stock would bear a different rate of interest and have a somewhat more uncertain value. Common stock might show returns varying from zero to an excessive return on the investment. By deducting the value of the plant from the total value of the developed power, all the uncertain factors incident to the marketing of the various classes of securities are eliminated from consideration.

The cost of building a plant in any given location can be closely estimated; the operating cost of such a plant can also be closely calculated. Assuming a ready market at an established price, three definite factors, considered with the amount of power available, are all that is required to produce an accurate result.

The operating costs then will be:

Operators (equivalent to 4 men per annum)	$7 800
Labor (" " 3 " " ")	4 380
Office labor and supplies	1 200
Oils, waste, and mechanical supplies	750
Taxes, 1½%, assessed on $863 940	12 959
Compensation insurance on labor	900
Repairs and renewals (breakage)	1 200
Depreciation on mechanical equipment, 4% on $278 000.	11 120
Depreciation on structures, 2% " 396 000.	7 920
Depreciation on excavation, 1% " 40 000.	4 000
Annual operating cost	$52 229

Insurance, other than compensation insurance on operatives, is eliminated, as there is nothing subject to fire loss in a modern hydro-electric plant, except the generators, and they are non-insurable.

Repairs and renewals are taken to mean breakages, as otherwise they tend to offset depreciation and could be carried to the extent of practically eliminating depreciation except by obsolescence. Excavation work is depreciated by obsolescence only.

This will give net earnings as follows:

Gross annual income	$205 800
Annual operating cost	52 229
Annual net earnings	$153 571

Capitalization

Equitable valuation by the method of capitalized net earnings is dependent on the rate used, and this is governed by the surrounding circumstances.

Generally speaking, 10% is a rate that is acceptable to parties financing a water-power project, as representing a fair return on invested capital. This rate is ordinarily accepted by the Courts.

Thus, the value of the water rights becomes:

Annual net earnings, \$153 571, capitalized at 10% = value developed	\$1 535 710
Deducting cost of development	863 940
Value of undeveloped water rights	\$ 671 770

The value per horse-power of undeveloped water rights, based on the gross horse-power, would be \$74.64; based on the quantity of power deliverable to the market (5 945 h.p.), the value per horse-power would be \$113.00.

Since 1917 the writer has made more than seventy-five water-power appraisals which, in instances, have been submitted to the New York State Public Service Commission and to the income tax units of the United States Treasury Department. The method submitted is the result of debates with the engineers acting for these parties preliminary to arriving at a common ground of agreement.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE GREAT YANGTZE BAR

By H. von Heidenstam,* M. Am. Soc. C. E.

Synopsis

The object of this paper is to describe the engineering investigations of the Yangtze Estuary for purposes of improving navigation to and from the Port of Shanghai, China. Since the opening of the river to foreign trade in 1842, the volume of shipping entering it has grown enormously, now amounting to about 16 000 000 British registered net tons per year. Within recent years the draft of ships has increased to a figure at which navigation over the bars of the Yangtze has become difficult. The interests of the Yangtze ports in general and of Shanghai in particular call for improvements.

The engineering problem is one to daunt the most courageous. The bars are many miles in length and the channels 2 or 3 miles wide. The depth in the main channel is 19 ft. at ordinary spring low water, 27 ft. at neap high water, and 31½ ft. at ordinary spring high water. The material is mud, and the bars are bordered by enormous shoals of similar mud, some of which shift and re-form constantly. The water is always heavily charged with silt and while there is a fresh-water discharge perhaps exceeding in maximum value that of any other river in world, its effects are practically neutralized in the estuary by a prodigious tidal flow.

The changes caused by tidal forces in the course of a few days are sometimes greater than from the largest conceivable excavating machinery in many months. The site is fully open to the sea. The main channels, separated by shoals, have cross-sections of about 1 000 000 sq. ft., widths of 3 to 4 miles, and depths in their shallowest sections over the bars of about 19 ft. at ordinary spring low water. There is no solid foundation for training works and their magnitudes would be appalling from a financial point of view. Under all circumstances a record of the data collected cannot but be of general interest.

Note.—Written discussion on this paper will be closed in February, 1928.

* Capt., Royal Swedish Corps of Engrs.; Engr.-in-Chf., Whangpoo Conservancy Board, Shanghai, China.

A summary is given of the investigations made by the writer, for the Whangpoo Conservancy Board, since 1914, culminating in a special investigation, from 1919 to 1921, as to the possibilities of improving the approaches to, and the facilities in, Shanghai Harbor. This investigation was concluded by the convening of an International Committee of Consulting Engineers, consisting of seven river and harbor experts, who reported at the end of 1921. The principal recommendations made by this Committee included the dredging of the South Channel and the construction of a system of public wharves in the Whangpoo River. This report is still under consideration by the Central Government.

Some of the difficulties involved in the dredging proposal are noted. An Appendix is also attached, dealing with the principal technical data collected.

Introduction

The problem of improving the sea approaches to Shanghai as here described is most difficult. Shanghai is situated on the Whangpoo River, about 15 miles from its mouth, the river itself forming the harbor (Fig. 1). The Whangpoo enters the estuary of the Yangtze about 30 miles above the main bar of that river, the "Fairy Flats". The depth on the crest of that bar at extraordinary low water is only about 16 or 17 ft.; at ordinary spring low water, 19 ft.; at neap high water, 26 to 28 ft.; and at ordinary spring high water, 30 to 32 ft. There are two other entrances to the Yangtze, but one of these is quite beyond improvement, while the other is similar to the Fairy Flats and more indirect.

The shipping visiting Shanghai has now increased to such a volume that, reckoned by tonnage, this city is among the first six ports of the world. Moreover, the drafts of the ships are steadily increasing. Many ships having drafts of about 30 ft. now arrive and an increase in draft is certain within a few years. A vessel of 30-ft. draft is delayed only occasionally, but those of larger draft would have great difficulty. The deepening of one of the bars, therefore, is indispensable to the future welfare of the Port of Shanghai.

The magnitude of the operation is in itself appalling. Training works of accepted types and of an extent proportionate to the size of the bar might easily cost $50 000 000 (gold). Even so they would not be of such stability as to assure undeniable success. Dredging is only conceivable if the very largest and most efficient modern machines are to be used. The mere surveying and marking out of the site of the works is a very large operation. All these circumstances enhance the technical interest of the investigations already made and the professional opinions given.

History and Record of Reports

In 1876, Messrs. J. de Rijke and G. A. Escher, two Dutch civil engineers employed by the Japanese Government, were requested by the Shanghai Consular Body to report on the Inner Bar of the Whangpoo River (which has since been eliminated by the works of the Whangpoo Conservancy Board).

A supplement to their report included an "Essay on the Fairway from and to Shanghai through the Estuary of the Yangtze".

At that time it appeared from a comparison of the British Admiralty charts issued since 1842 that the South Channel (Fig. 1) was becoming shallow. More recent maps show that this tendency, if at all a fact, ceased subsequently. The authors of this 1878 report believed that the North Channel (Fig. 1) was growing at the expense of the South Channel and that by closing various small crossings and skillfully cultivating a channel between the shoals opposite the mouth of the Whangpoo a good passage to the North Channel could be developed.

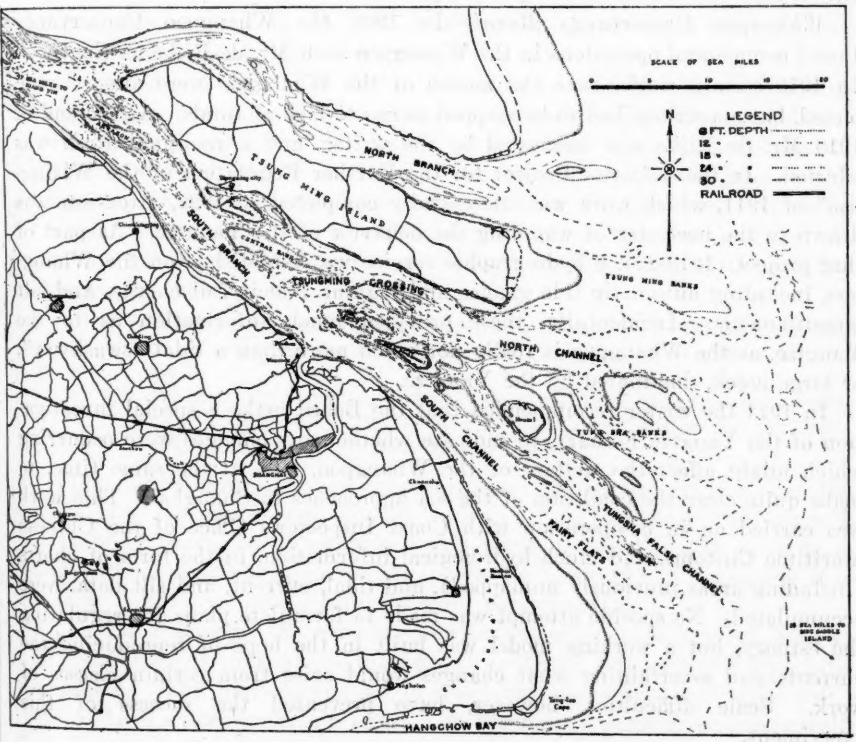


FIG. 1.—GENERAL PLAN OF THE YANGTZE ESTUARY.

It should be noted that the outer bar in the North Channel is usually deeper than that in the South Channel, the real obstruction being an upper bar (Tsungming Crossing). They had in mind a low-water draft of only 15 ft. In their opinion mechanical dredging was impracticable:

"* * * even for assisting only it must prove too expensive in so extensive an estuary. The dredgers, too, as well as the mud scows, would require to be built in a specially costly manner to fit them for navigating rough and dangerous water; but even if they were so constructed, still the number of off days would be very great."

Again,

"Everybody will also understand that any attempt to dredge the estuary of the Yangtze would be throwing money away, and that a single storm would destroy many months' work."

This very definite opinion given in 1876 is probably still true with respect to bucket dredges (which are distinctly suggested by the reference to scows), in spite of the great increase in capacity of dredges. It may not be true, however, of modern suction dredges some of which have a very large output and, by the use of hoppers or pipe lines, become independent of a scow service.

The report has interest as giving a word of warning to Shanghai, but has only a limited bearing on the present problem.

Whangpoo Conservancy Board.—In 1906 the Whangpoo Conservancy Board commenced operations in the Whangpoo with Mr. de Rijke as Engineer. In 1910 certain works near the mouth of the Whangpoo were almost completed, but operations had to be stopped owing to lack of funds. At the end of 1910 Mr. de Rijke was succeeded by the writer, and a re-organization was effected. In the writer's "Project for the Further Regulation of the Whangpoo" of 1911, which work was successfully completed in 1922, attention was drawn to the necessity of watching the behavior of the Yangtze. As part of that project (1912-22), a hydrographic service was established on the Whangpoo, including automatic tide gauges and regular velocity, discharge, and silt measurements. Incidentally, these provided much information as to the Yangtze, as the Whangpoo is really not much more than a tidal "swashway", or large creek, dominated by the Yangtze.

In 1914 the writer recommended that the Board make a special investigation of the Yangtze Estuary, to indicate whether any changes were occurring which might affect the welfare of the Whangpoo, and, at the same time, to make quite clear the condition of the sea approaches to Shanghai. This work was carried on in co-operation with Coast Inspector's Office of the Chinese Maritime Customs, and much hydrological information, in the form of charts (including areas previously unmapped), and tidal, current, and silt data, were accumulated. No specific attempt was made to formulate plans for regulating the estuary, but a working model was built in the hope of reproducing the currents and ascertaining what changes would arise from certain classes of work. Scale difficulties, however, have prevented the success of this experiment.

As a result of this investigation, and in order to consider future needs, it was decided to take up the general question of possible improvement. In 1917, by request of the Board, the Vattenbyggnadsbyran, Consulting Engineers, of Stockholm, Sweden, and the writer, submitted a "Report on the Future Development of Shanghai Harbour", which analyzes Shanghai's position in regard to trade routes and includes a definite technical consideration of the approaches. A "Memorandum on a 'Port de vitesse' for Shanghai on, or Canal to, the Hangchow Bay" (see Fig. 1) was appended.

Shanghai Harbor Investigation.—A great deal of interest was created by the publication of this report (of which the definite recommendation was a

further and full inquiry), and, in 1919, the Board authorized and commenced a "Shanghai Harbor Investigation" which was to include as complete a survey as possible and, in addition, projects for improvement of the approach and the construction of harbor works. The work was carried on in three sections under the writer's direction, as follows:

Section I.—Surveys of estuary, etc., in charge of E. C. Stocker, M. Am. Soc. C. E.

Section II.—Harbor designs, in charge of Mr. C. W. Simon.

Section III.—General studies and secretarial, in charge of Mr. H. Chatley.

The delicacy of the approach problem was felt to be such that in addition to the Shanghai investigation two consulting engineers were retained, Mr. C. A. Jolles, of Arnhem, Holland, and Vattenbyggandsbyran (The Hydraulic Engineering Bureau), of Stockholm.

It was part of the duties of the latter firm to consider also the desirability of making a new approach by means of Hangchow Bay (thus avoiding the present bars altogether). Similarly, Section I was to make (in co-operation with the Chinese Maritime Customs) a full survey of the Hangchow Bay (Fig. 1). This survey revealed that the depths in this Bay were just sufficient for the largest drafts likely to visit Shanghai. Thus, the advisability of adopting this method of avoiding the obviously great difficulties of improving the Yangtze depended on two questions:

(*a*) Whether Hangchow Bay was likely to maintain its present depths, or, if not, whether a deep channel could be artificially maintained? and,

(*b*) Whether constructing a first-class ship canal or, as an alternative, a new coast "port de vitesse", with good water connections to Shanghai, was economically practicable?

The Vattenbyggnadsbyran expressed the very definite opinion that Hangchow Bay was slowly but steadily silting up. The survey and comparison with other charts partly confirmed this opinion. As the Bay is quite open and without continuous current channels the possibility of deepening it artificially if required in the future seemed very doubtful.

In the matter of the connection with Shanghai, a ship canal with locks of the Panama Canal type to exclude salt, silt, and tide appeared prohibitively expensive. The "port de vitesse" in itself was also a matter of great cost and would have a very uncertain relation to Shanghai's peculiar economic welfare.

The Vattenbyggnadsbyran expressed the opinion that the "idea of reaching Shanghai *via* the Hangchow Bay should be definitely given up".

Mr. Jolles, with his long experience on Dutch waterways, was asked to report on the estuary problem only, and expressed the opinion that the bar was largely due to coagulation of the silt by saline flood currents; that the South Channel was the only one for which regulation should be attempted; and that training works would be prohibitively expensive and uncertain of success, but that dredging might be successful. He estimated that 25 000 000 cu. yd. of material might be involved. No "streaming away"* need be anticipated as the silt

* Removal of mud in excess of the quantity dredged by reason of the current acting on the disturbed bottom.

precipitation will not undergo any change. Drag suction dredges might be the only suitable type. He finally suggested dredging work in the channel connecting the "Tungsha Channel" to the South Channel (southwest of House Island, Fig. 1) and in the Tungsha Channel itself, to a depth of 25 ft. below low low water, involving about 12 500 000 cu. yd. of dredging and 900 000 sq. yd. of mattress protection. The cost would be about 22 000 000 Taels (say, $15 000 000); the permanence would be uncertain.

The Tungsha Channel (Fig. 1) is a relatively recent development. The South Channel was formerly wider and then was split by the shoal now termed the Tungsha Spit (Fig. 2). The Tungsha Channel is really part of the former South Channel and for some years it was expected to become eventually the navigation route, usurping the place of the narrowed course of the South Channel. In recent years, however, it has shown no tendency to increase in depth and the bar at its head (which Mr. Jolles proposed to dredge) has shallowed.

FIG. 2.—THREE LAYOUTS FOR DREDGING PROJECTS IN SOUTH CHANNEL, YANGTZE RIVER.

The Vattenbyggnadsbyran in its report make an elaborate but rather academic study of the shoal movements and concluded that the only permanent tendency is for the deepest channel to swing southward. It held, therefore, that the present South Channel is the most stable of all and definitely recommended dredging along the axis of that channel.

On the basis of the local investigation and studies, the writer, prior to the arrival in Shanghai of the International Committee of Consulting Engineers in October, 1921, prepared a general report on the entire problem (the harbor itself and its approaches) for the use of the Committee, which included plans and estimates for various solutions.

In regard to the estuary, it put forward three alternative dredging projects in the South Channel (Fig. 2):

(1) A cut, 11 miles long, along the thalweg of the Tungsha Channel.
(2) A direct diagonal cut, 9 miles long, between the two channels.
(3) A direct cut, 20 miles long, along the axis of the South Channel.

The net volumes of dredging in the three cases were, respectively: (1) 13 400 000 cu. yd.; (2) 12 500 000 cu. yd.; and (3) 22 000 000 cu. yd.

Suitable harbor works, survey, and dredging craft were included, with five years' operation on whichever of the three layouts might be preferable, estimated to cost about 8 325 000 Taels, or, say, $6 000 000.

Report of International Committee of Consulting Engineers.—As already mentioned, purely tentative schemes covering all reasonable possibilities were submitted to and studied with great care by the International Committee of Consulting Engineers which met in October, 1921. Its members were: Messrs. W. M. Black, Major-General (*Retired*), U. S. Corps of Engineers, M. Am. Soc. C. E. (United States); I. Hiroi (Japan); P. G. Hornell (elected by the Chinese Chamber of Commerce); F. Palmer (Great Britain); P. J. Ott de Vries (Holland); L. Perrier, Ingénieur en Chef, Corps des Ponts et Chaussèes (France); and the writer (Chairman).

The Committee's joint report published at the end of 1921, made the following definite recommendations:

In Regard to Approaches:

General Scheme:

(*a*) Selection of Approaches:
That the South Channel lends itself most readily to improvement and that the work of obtaining a greater depth should be undertaken along this channel.

(*b*) Regulation:
That deepening in the South Channel should be undertaken by dredging only, at least in the first instance, and that by this means there is a reasonable prospect of permanent improvement.

(*c*) Observation:
That extensive and continuous observations should be carried on in the estuary to provide further information for river engineering purposes, in addition to that available from the navigation charts which may be published from time to time by the Chinese Maritime Customs.

In Regard to Immediate Requirements:

(*a*) To carry out forthwith the river engineering observations indicated under the General Scheme.

(*b*) To make and maintain a channel through the Fairy Flats, deepening it gradually foot by foot, to as great a depth and width as required and possible, and to provide as soon as feasible a channel of 600 ft. bottom width for the passage of ships drawing 33 ft. at ordinary neap high water.

The estimate for the work, under "Immediate Requirements", for the improvement of approaches, was as follows:

Capital Expenditure:

Survey plant	Shanghai Taels		200 000
Dredging plant	"	"	4 200 000
Incidental plant	"	"	125 000
Total	Shanghai Taels		4 525 000

Annual Expenditure:

Survey	Shanghai Taels		100 000
Dredging	"	"	1 000 000
Total	Shanghai Taels		1 100 000

This plan is still under consideration by the Central Government and its adoption involves an agreement between that Government and the Diplomatic Body of Foreign Representatives as to the constitution of the Harbor Authority.*

The Present Project†

Survey.—It will be observed that the actual location of the attempted deepening has been left open, with the idea that further and repeated survey work would throw more light on the position of the deepest channels and the stability of the bed masses. This is not so easy as appears at first sight. The river is too wide to permit very exact re-section; permanent beacons can only be erected on the south shore and on the islands west of Tungsha Bank; the depths vary only gradually; the bottom varies in softness; strong currents affect the soundings; and tidal levels are difficult to compute with the necessary exactitude.

To date, the tides on the bar have been determined by reading on a series of poles at Nanhui on the South Bank (Fig. 1) and also by interpolation between an automatic gauge at Side Saddle Islands and another at Woosung, or by temporary poles on some of the islands; but owing to the distance from shore the tide pole at Saddle Islands has only been "leveled" by computation from water slopes—similarly with the tide poles on the islands. The tide poles at Nanhui have been connected by levels with Woosung, but it is not certain whether the high waters across the gently sloping shore out to the bar there are really horizontal.

Soundings have also been taken on a lightship, which is anchored over the Fairy Flats and swings over a moderately flat bottom. It is obvious that the accuracy of the tidal rise at the time of sounding the bar is only to the nearest foot, and since the soundings themselves by reason of currents, softness of mud, etc., are probably uncertain by a similar amount, the actual levels of the bottom (Figs. 3, 4, and 5) are in doubt by as much as perhaps 2 ft.

---

* No criticisms have yet been made by outside parties on the estuary project, except a paper by Mr. Canter Cremers, in the September, 1923, issue of *The Dock and Harbour Authority* (London), in which he expresses the fear that the inward running flood currents will neutralize the dredging by reason of their excess of force on the bed over the ebb currents, due to salinity.

† As outlined by the International Committee of Consulting Engineers, 1921.

Over a large area of the bar the computed values differ only by 1 or 2 ft. in distances of ½ mile. The top, therefore, is practically level, and it is a question whether there is any definite thalweg across it. Longitudinal profiles of the South and North Channels are shown in Figs. 3 and 4. Fig. 5 gives a cross-profile of the estuary at the shallow part (Fairy Flats). Eleva-

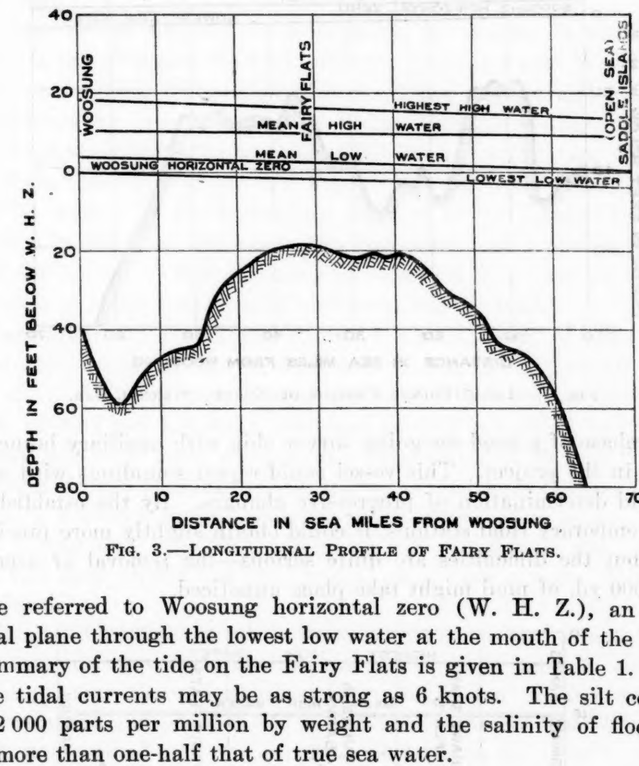


FIG. 3.—LONGITUDINAL PROFILE OF FAIRY FLATS.

tions are referred to Woosung horizontal zero (W. H. Z.), an imaginary horizontal plane through the lowest low water at the mouth of the Whangpoo.

A summary of the tide on the Fairy Flats is given in Table 1. At spring tides the tidal currents may be as strong as 6 knots. The silt content may rise to 2 000 parts per million by weight and the salinity of flood currents may be more than one-half that of true sea water.

TABLE 1.—STANDARD WATER LEVELS AT FAIRY FLATS (NANHUI BEACON).

	Water level referred to W. H. Z., in feet.	Rise, in feet.	Depth on crest, in feet.	SEASONAL CORRECTION TO BE APPLIED FOR:	
				January.	July.
Highest high water	18.0	21.8	37.8	......	......
Standard spring high water	11.8	15.6	31.6	−0.33	+1.35
Mean high water	9.6	13.4	29.4	−0.62	+1.24
Highest low water	8.6	12.4	28.4	......	......
Standard neap high water	7.0	10.8	26.8	−0.10	+1.27
Mean water level	5.4	9.2	25.2	......	......
Standard neap low water	3.6	7.4	23.4	−1.22	+0.89
Mean low water	0.8	4.6	20.6	−0.30	+0.81
Lowest high water	0.2	4.0	20.0	......	......
Standard spring low water	− 0.0	3.8	19.0	−0.28	+0.28
Lowest low water	− 3.8	0.0	16.0	......	......

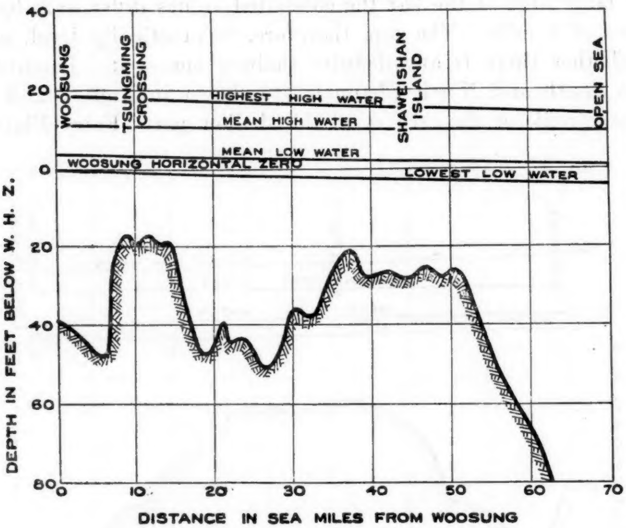


FIG. 4.—LONGITUDINAL PROFILE OF NORTH CHANNEL BAR.

The purchase of a good sea-going survey ship with auxiliary launches, etc., is included in the project. This vessel could repeat soundings with a view to a more rapid determination of progressive changes. By the establishment of numerous temporary tidal stations, it could obtain slightly more precise measurements; but the difficulties are quite serious—the removal or accretion of even 1 000 000 yd. of mud might take place unnoticed.

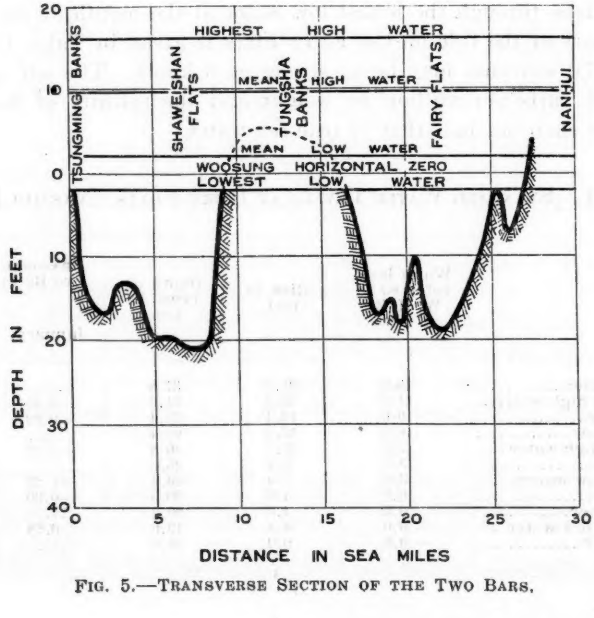


FIG. 5.—TRANSVERSE SECTION OF THE TWO BARS.

It will probably be necessary also to erect one or more permanent tide gauges (automatic), but these will have to be very massive and well founded. The laying of buoys for dredging will also be part of the survey work, so that the vessel must be fitted with cranes, etc. It will need a speed of 10 knots to make headway against the current and must have auxiliary motor craft for shoal work.

Work Base.—The distance from Woosung to the bar is about 30 miles. While this is not excessive for high-powered craft, it would be very desirable to have a nearer base for residence, stores, light repairs, and storm shelter. In the preliminary project submitted to the Committee there was included a design for a work harbor to cost about 500 000 Taels at House Island (about 10 miles from the center of the bar, Fig. 1), but this was intended principally for mattress work. As the Committee inclined to the opinion that dredging alone should be tried, no allowance has been made for a new work base, and the dredges will have to visit Woosung or be served by a special collier capable of bunkering at a mooring in such shelter as can be found.

This question calls for careful study. On a conservative basis the annual output of each dredge should be 5 000 000 cu. yd., or, say, 20 000 cu. yd. per working day. Allowing even a very low rate of 1 h.p.-hour per yd. shifted and 3 lb. of coal per h.p.-hour (includes getting up steam, etc.), the daily coal supply will be at least 30 tons, and more probably 50 tons. The largest drag suction dredge familiar to the writer is the *New Orleans* (Frühling), which has a 3 000-yd. hopper and 300-ton bunkers. For reasons of draft a 2 000-yd. hopper and, say, 200-ton bunkers will probably be the limit for this work, so that each dredge will have to coal about once a week. Oil fuel might be considered.

Dredging.—The Committee's recommendations are very simple. By means of the largest dredges procurable an attempt is to be made to cut a channel through the Fairy Flats, deepening it "gradually foot by foot to as great a width as required and possible and to provide as soon as feasible a channel of 600 ft. bottom width" and "a deepening of 9 ft. on the crest". The reasons leading to the momentous decision in favor of the Yangtze Estuary Approach as compared with the Hangchow Bay Canal were very complex, with various political, financial, and technical bearings.

Choice of Fairy Flats Channel.—The Committee did not indicate exactly in what position in the South Channel the dredging should be done, believing that this was a matter to be decided from the results of a continuous survey of the bar, which should reveal the most favorable location. However, it suggested that a cut directly over the Fairy Flats in the South Channel offered the best prospect of success.

The writer concurs in the decision purely from a technical point of view, for the following reasons:

(1) An effort of some kind must be made to improve the approaches which, at present, limit the draft of shipping at Shanghai. Investigation having shown that effective training works would be prohibitively expensive, dredging is the only possibility.

(2) The center line of the South Channel is at present the most direct navigable course for the majority of the shipping, so that, other things being equal, it is the most convenient location.

(3) As to physical shape, the bed of the Fairy Flats is simpler than that of any other passage, and, therefore, offers the best topographical conditions for improvement.

(4) The Fairy Flats, lying in the general extension of the Lower Yangtze, offer better probabilities of direct scour by the ebb.

(5) Historical evidence shows that permanent shoal formation has occurred more continuously about the mouths of the other two openings rather than about the entrance to the South Channel, which, therefore, is the preferable location from the standpoint of secular change.

(6) Current measurements show that the ebb on the Fairy Flats is as strong as, if not stronger than, that in the other passages. Hence, this site offers the prospect of maximum natural scour and of minimum re-accretion.

(7) Soundings show that from time to time depressions of large area but small depth occur on the axis of the Fairy Flats and persist for some years. This indicates that the bottom is in a state of labile equilibrium, with occasional inclination to scour. The quantities of material shifted in the formation and re-accretion of such depressions are far greater than any reasonable dredging project could cope with, but the fact that such variations of depth do occur suggests that a continuously applied effort on a small area along the line of the strongest currents may co-operate with the scouring tendency of the ebb currents to produce a permanent depression.

(8) These considerations taken conjointly give a reasonable hope that an artificial deepening over a relatively minute fraction of the width of the Fairy Flats would have such a measure of stability as to warrant the attempt to create and maintain it.

Proposed Method of Dredging.—There are two outstanding difficulties: (*a*) The quantity to be removed, which is in the neighborhood of 20 000 000 cu. yd.; and (*b*) the re-accretion that may occur.

There can be little doubt that the slow excavation of a trench to the full depth would form a dead-water pocket in which accretion would be extremely rapid, and a method of dredging that provides a long small cut seems much to be preferred. Since the degree of stability of the sides of the excavation is small, a broad, shallow cut has apparent advantages over a narrow, deep one. On the Taku Bar, near Tientsin, some success has been found with the latter form. So far as the writer is aware the drag suction dredge is the only type that gives the desired results. Ordinary suction dredges (with rotary cutters), or bucket dredges, would not serve the purpose since they cannot work well unless they make a deep and wide lateral cut, the mere volume of which makes rapid longitudinal progress impossible. Such progress is indispensable in order to encourage scour.

Other things being equal, a pipe-line discharge to a point beyond the dredged channel would be preferable, but the marine conditions and the speed required appear to prohibit it. Overboard discharge is open to the objection that part of the mud will be re-deposited over the area of the cut and thereby partly if not wholly undo the work.

The marine conditions suggest that a self-contained hopper dredge will be enabled to work more continuously than a hopper barge arrangement. The decision, therefore, seems to rest exclusively with the drag-suction, self-propelled, hopper-type dredge.

The desired cut increases in length as it deepens, to about 25 miles, but if the shorter shallow depression maintains itself, the stability should increase with the depth, so that the practical question will be rather one of rapidly channeling the bar over a length of, say, 10 miles. If the dredging speed is 3 knots (over the bottom) this means that one through cut might be made in a tidal period (allowing half the time for dumping). If an appreciable result is to be produced in 5 years the annual dredging capacity should be at least 5 000 000 cu. yd., or, say, 2 000 effective cu. yd. per hour (for 2 500 working hours); allowing only one-third the working time for actual dredging, this means a continuous capacity of 6 000 cu. yd. per hour, which would require the largest and most efficient machines yet built. Failure to achieve an output of this size will greatly reduce the possibilities of success unless the natural scour proves very favorable.

The brief details given show the extraordinary uncertainty and difficulty of the problem. Its magnitude appeals to the imagination of every engineer. The Appendix gives further data relating to the estuary. Perhaps other members can throw light on a possible solution.

APPENDIX

General Data

Historical Changes.—Owing to the inaccuracy of early maps there are no precise records of the undoubtedly great changes that have occurred in historic times.

Fig. 6 shows a series of rough maps compiled from old data, which clearly indicate the superior stability of the present South Channel, and the progressive deterioration of the North Branch. The various surveys made since the middle of the Nineteenth Century, the accuracy of which has steadily increased, do not give any distinct evidence as to systematic change in the two main bars; accretion seems to have occurred principally in the regions of the North Branch and Yangtze Cape. Erosion appears to be a purely local phenomenon initiated at concaves.

Tides.—The whole estuary is subject to tidal action. Tidal observations have been made for prolonged periods at Side Saddles (Fig. 7), an island at the extreme north of the Chusan Archipelago in the open sea just off the mouth; at Woosung Forts (Fig. 8), at the entrance to the Whangpoo; and at Kiangyin (Fig. 9), the hood of the estuary. Other places have been observed for shorter periods. The annual change of "stage" in the river reduces the tides in the upper part of the Estuary, as shown in Fig. 9. At Kiangyin the mean level has a fortnightly swing.

Tidal Currents.—The currents are quite strong. Maximum flood and ebb velocities of more than 6 ft. per sec. have been observed at a point about 20 miles above Woosung; and on the bar the maximum filament flood velocities at spring tides may be 10 ft. per sec. Outside the estuary there is a distinct clockwise rotary effect and even in the lower parts of the estuary (for example,

on the Fairy Flats) the tidal currents move in long flat loops so that there is a transverse (northward) current at slack water.

The non-tidal gradients in the estuary are very small (less than 1 ft. in 10 sea-miles), but the slopes of the simultaneous longitudinal water profiles may be as much as 1 ft. per sea-mile.

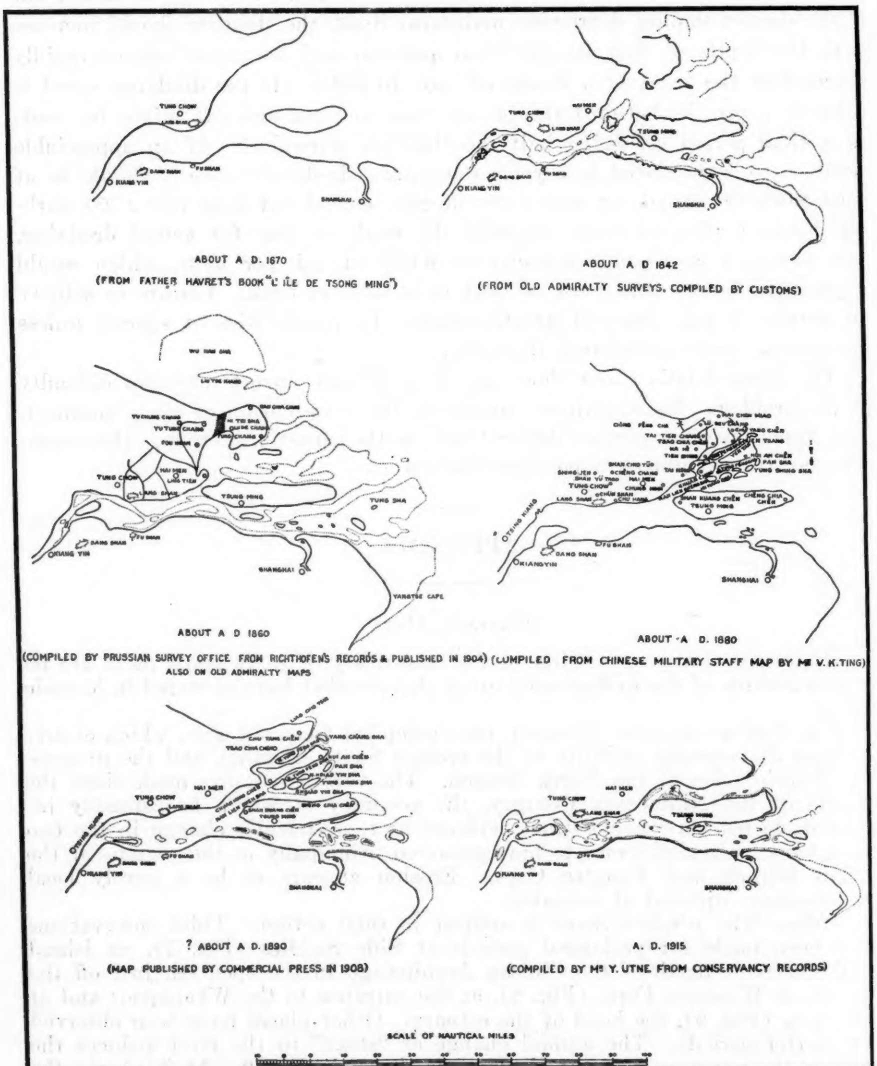


FIG. 6.—SERIES OF HISTORICAL PLANS OF THE ESTUARY.

Velocities.—Owing to the magnitude of the stream the observation of current velocities has not been very exhaustive, but it has been found that in most places the velocity near the bottom is about 60% of the mean velocity and that the mean velocity occurs at about 60% of the depth.

There are great irregularities in the velocity at different depths due to the lack of symmetry of the sections and the tidal-current reversals. The distribution across the stream shows nothing unusual in spite of the magnitude of the sections.

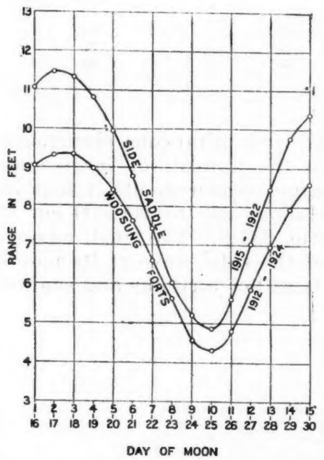


FIG. 7.—MEAN TIDAL RANGES THROUGHOUT THE LUNAR MONTH AT WOOSUNG AND SIDE SADDLE.

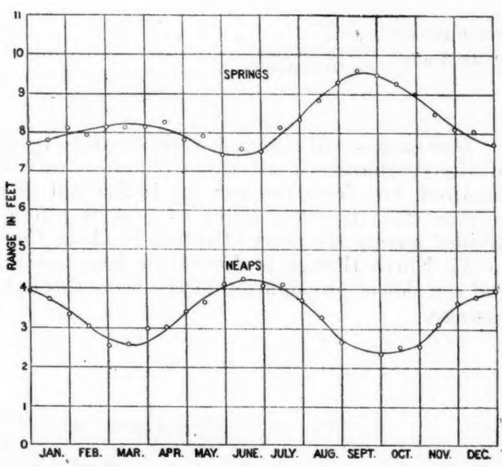


FIG. 8.—VARIATIONS IN TIDAL RANGE AT WOOSUNG FORTS.

Section Areas.—The sectional area of the whole channel (including all mouths) measured below low low water increases from about 250 000 sq. ft. at Kiangyin to more than 1 500 000 sq. ft. at the coast margin (above the bars).

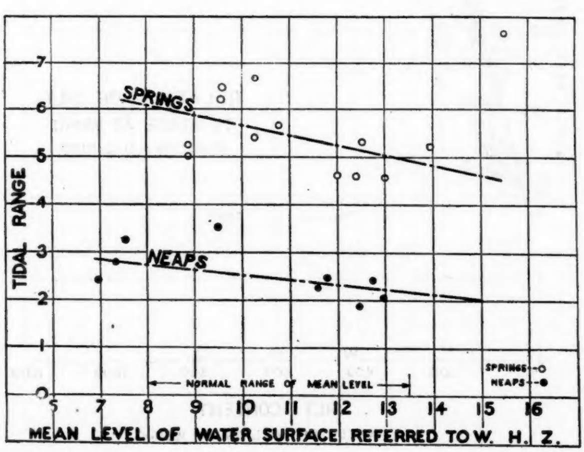


FIG. 9.—TIDAL RANGE AT KIANGYIN, WITH RESPECT TO MEAN GAUGE HEIGHT.

The sectional area below standard spring high water at the coast margin is about 3 000 000 sq. ft.

These areas are divided between the three entrance channels approximately, as shown in Table 2.

TABLE 2.

Channels.		Area below low low water, percentage.	Area below high water, spring tide, percentage.
North Branch		30	33
South Branch	North Channel	40	35
	South Channel	30	30

Discharges.—It has not been feasible to make a simultaneous observation of the discharge of all three channels owing to the number of large craft required, but from the various individual discharge measurements it appears certain that the total influx of a spring tide is about 250 000 000 000 cu. ft. divided among the three channels in about the ratio, 2:3:3. The small amount in the North Branch is due to the later arrival of the tidal wave at its mouth and the slower propagation of the wave through its rather tortuous and shallow passage.

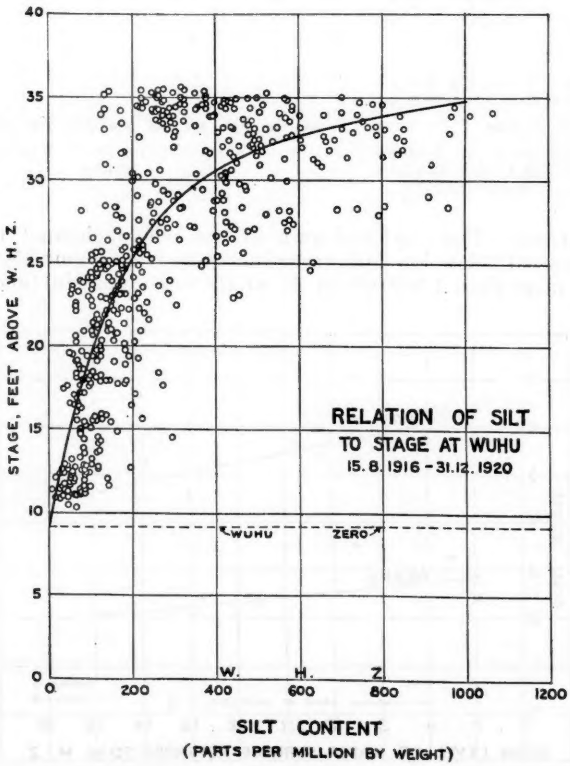


FIG. 10.

Tidal Magazine.—The capacity of the estuary is, of course, equal to the influx and diminishes for spring tides from about 200 000 000 000 cu. ft. at the coast line to 5 000 000 000, or less, at Kiangyin.

Run-Off.—From gaugings in the upper river it has been found that the fresh-water discharge of the river varies from about 250 000 cu. ft. per sec. to

more than ten times that amount, averaging about 1 000 000. The maximum is in the late summer and the minimum, in January.

Silt.—Immense quantities of silt are brought down. Fig. 10 shows the silt charge at Wuhu (a point where tidal influence is very small, about 300 miles from the mouth), in relation to the stage of the river, which, of course, varies with the run-off.

In the estuary the tidal currents disturb and keep in suspension a great quantity of silt so that even in the winter, when the supply of fresh silt from the interior is small, the spring tides working in shallower channels develop more velocity and the water has a maximum of turbidity.

Fig. 11 shows the variation of silt with the tidal range at a point in the Whangpoo (Pheasant Point), quite close to the Yangtze.

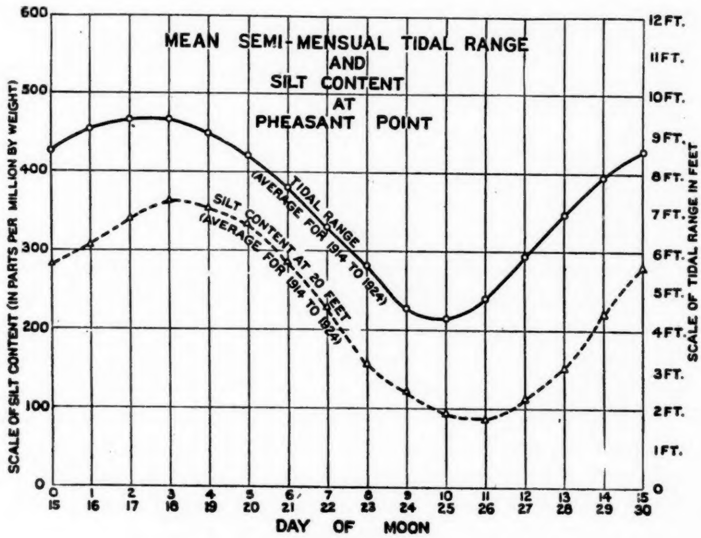


FIG. 11.

A few silt samples have been taken on the Fairy Flats and show values from one to three times as great as those taken simultaneously in the mouth of Whangpoo. High salinity is also observed there (0.5% to 2.0% which compares with 2.5% for the China Sea).

Geological Structure.—Borings have been taken on land and under water in the Estuary and show that below Woosung the general mass is alluvial mud, with occasional patches of fine sand, to a depth greater than any works are likely to extend. Quartzite or porphyrite islands occur outside the Estuary, but the nearest (Ariadne Rocks, Fig. 2) is a long way from the bars.

No evidence has been found as to the subsidence or elevation of the delta by subterranean causes or by sedimentation load.

Consistency of the Mud.—A great deal of consideration has been given to the nature of the mud and its variation. In the Whangpoo, except in the case of recent accretion, it is a heavy material containing sufficient colloidal matter to make it very firm; but it cannot be regarded as a real clay. Silica predominates as the mineral constituent, and the mud is fairly easily disturbed by a stream of water; but when consolidated it presents considerable resistance to dredging by grabs or bucket dredges. The mud on the Fairy Flats closely resembles that in the Whangpoo.

Meteorological Data.—Ample meteorological data are available. The Fairy Flats is practically in the open sea, being sheltered only on the southwest. The North Channel Bar is quite in the open. Northwest winds prevail in the winter and the bar is quite choppy during flood current. Southeast winds prevail in the summer and ebb currents are then the more disturbed. Very dangerous typhonic winds may occur in the late summer and autumn. Moderate or smooth weather occurs for about 80% of the time. About 100 hours of fog per year occur on the Fairy Flats and about 400 on the North Channel Bar.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

SIDE SPILLWAYS FOR REGULATING DIVERSION CANALS

By W. H. R. Nimmo,* Assoc. M. Am. Soc. C. E.

Synopsis

In 1922, the Hydro-Electric Department of the Government of Tasmania completed a diversion channel of 450 sec-ft. capacity to convey water from the Upper Ouse River, at Liawenee, to the Great Lake storage. The intake of this channel is in a deep and narrow gorge where a low weir, only about 100 ft. long, suffices to divert the normal flow into a flume, ½ mile in length, built on a rock bench excavated along the wall of the gorge. From the outlet of the flume a canal, part of which is excavated in earth, conveys the water a farther distance of 5 miles where it is discharged into the Great Lake.

Within the catchment area are a large number of snow-fed lakes and lagoons and the stream is very flashy, the flow sometimes increasing from a few hundred to 12 000 sec-ft. within about 10 hours. The estimated maximum flood flow is 20 000 sec-ft. At present, the flow in the canal is controlled by a gate operated by a resident attendant but, as the country is inhabited only by a few shepherds in summer and is practically uninhabited in winter, it is desirable that some means of automatically regulating the flow in the canal should be devised. To be satisfactory in service any automatic arrangement must comply with the following conditions: (1) When the flow of the river does not exceed 450 sec-ft., all the flow must be allowed to pass into the concrete flume; and (2) when the total flow in the river exceeds 450 sec-ft., the flow in the concrete flume must not exceed 550 sec-ft. To meet the foregoing conditions several types of mechanically operated gates were considered, but those depending for their action on small differences of water pressures were discarded as they would fail to function during freezing weather. As the head-works are likely to become coated with ice for several weeks at a time, the forces applied to move the gate must be large, and for this reason gates operated by floats were found to be unsuitable. The writer then inves-

* Chf. Draftsman, Main Roads Comm., Brisbane, Queensland, Australia.

tigated the theory of flow in a side spilling channel with a view to devising a type of regulator without moving parts.

This paper presents the theory developed, describes the experiments carried out to test it, and the adaptation of the theory to the design of automatic head-works for the Ouse-Great Lake Canal.

Head-Works of the Ouse-Great Lake Canal

The head-works include a low concrete weir having a vertical up-stream face, a flat top, and a sloping down-stream face. At one end of the weir a converging channel, shown on Fig. 1, leads the water into the flume, this channel being bounded by the curved end of the weir on one side and by a concrete wall on the other. The crest of the weir is just high enough at all points to provide the necessary velocity head and entrance losses for the normal flow of 450 sec-ft. Down stream 184 ft. from the throat of the converging channel, is a head-gate 9 ft. wide, with vertical walls (visible in Figs. 5 and 9), which controls the entrance to the concrete flume. The outer guides of the gate are recessed so as to be flush with the walls of the flume and offer no obstruction, but the central guide consists of a 5 by 4½-in. by 18-lb., rolled steel joist.

Between the head-gate and the converging channel, there is a timber flume of trapezoidal section shown in plan, Fig. 1. Structural details of this flume are shown in Fig. 2, and Fig. 4 is a general view, looking up stream from the head-gate. Fig. 4 also shows the converging inlet and the weir, in which openings, now closed by stop-planks, have been left to carry the dry-weather flow during future alterations to the head-works. For a distance of 40 ft., up stream from the head-gate, the timber flume is on a curve of 198 ft. radius. The floor, which is flush with that of the concrete flume, has a normal width of 5 ft. 8 in. The side of the flume next to the river is intended to act as a spillway throughout the straight portion of its length, and the lip is 5 ft. above the floor, at which depth the water area is 45 sq. ft., being the same as that of the concrete flume, at its normal depth of 5 ft. The floors of both the timber and the concrete flumes are laid on a grade of 1 in 330. Along the entire side away from the river and on the curved portion of the side next the river, the wall of the flume is built 2 ft. above the lip of the spillway portion.

The throat of the converging channel has a width of 9 ft. and a depth of 5 ft., corresponding to the width and normal depth of flow in the concrete flume. At the down-stream end of the timber flume, a transition, 14 ft. long, connects the trapezoidal section to the rectangular section at the head-gate. In this transition the slope of the walls decreases uniformly and, consequently, their surfaces are slightly warped. At the up-stream end of the timber flume a similar transition, 12 ft. long, connects with the throat of the converging channel. Figs. 5 and 9, which were taken when the head-gate was closed, show the whole flow of the river—approximately 400 sec-ft.—discharging over the side of the timber flume. The concrete flume is 6 ft. deep, the free-board being 1 ft. at the normal flow of 450 sec-ft.

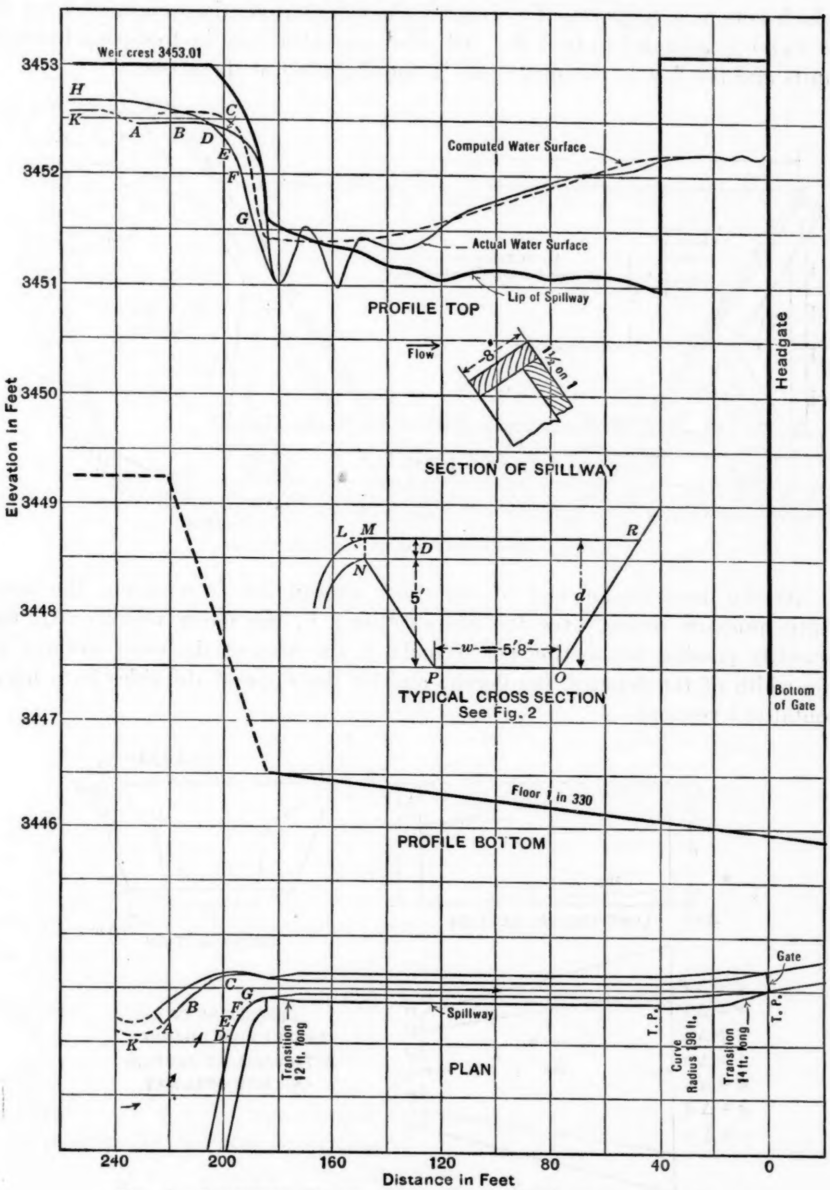


FIG. 1.—LIAWENEE FLUME EXPERIMENT NO. 19, GENERAL PLAN.

THEORY OF FLOW IN ANY OVERFLOWING TRAPEZOIDAL CHANNEL

Let Fig. 3 represent the plan, longitudinal section, and cross-section of a short length, Δx, of any trapezoidal channel, having varying bottom width and side slopes and also having water discharging outward over one side

which acts as a spillway. To simplify the calculations the weight of 1 cu. ft. of water is assumed to be 1 lb. All other quantities are in foot-pound-second units and the flow is assumed to be in the direction of the arrow.

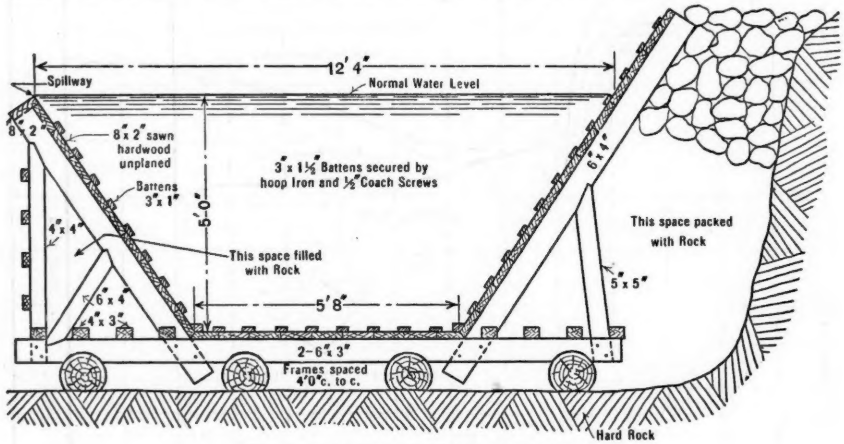


FIG. 2.—CROSS-SECTION OF TIMBER FLUME.

At the down-stream end of the short section, let P represent the total static pressure acting over the cross-section; V, the mean velocity; Q, the quantity passing the section per second; A, the area of the cross-section; w, the width of the floor; d, the depth; and let the slope of the sides be n horizontal to l vertical.

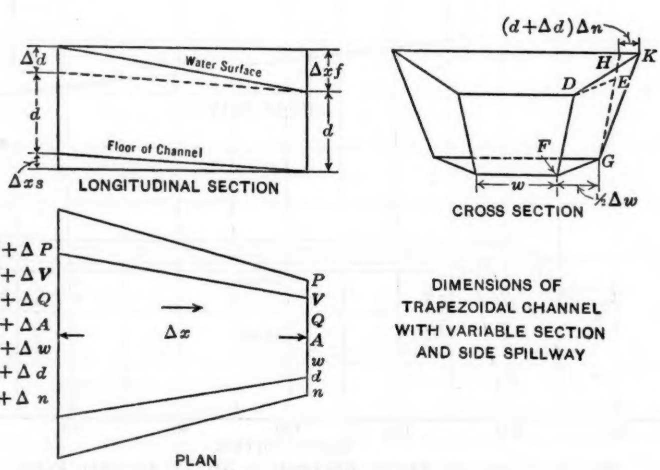


FIG. 3.

At the up-stream end of the section the corresponding quantities are to be denoted by $P + \Delta P$, $V + \Delta V$, $Q + \Delta Q$, $A + \Delta A$, $w + \Delta w$, $d + \Delta d$, and $n + \Delta n$, respectively. ΔQ, Δw, Δd, and Δn are positive, when Q, w, d, and n, respectively, are increasing up stream. Also, left f represent the loss

of head per unit length due to friction and s, the slope of the floor per unit length, s being positive when the floor of the channel slopes down stream.

Then, ΔQ represents the quantity of water which is discharged over the side of the channel in the length, Δx, and, as it retains its momentum parallel to the channel, a certain amount of momentum is lost with it. The value of ΔQ is given by the usual equation

$$\Delta Q = c\, \Delta x\, D^{\frac{3}{2}} \qquad \text{(1)}$$

in which, D is the head of water over the spillway lip, and c is the constant.

From the principle of conservation of momentum, the change of momentum in the length, Δx, less the momentum lost with the overflowing water, must equal the algebraic sum of the external forces. The external forces are:

(1) The static pressures on the end areas, given by,

$$P = \left(\frac{w\, d^2}{2} + \frac{n\, d^3}{3}\right) \qquad \text{(2)}$$

and,

$$P + \Delta P = \left(\frac{(w + \Delta w)\,(d + \Delta d)^2}{2} + \frac{(n + \Delta n)\,(d + \Delta d)^3}{3}\right) \qquad \text{(3)}$$

(2) The reaction from the projected area of the side-walls: The rectangle, $D\, E\, G\, F$, and the triangle, $H\, K\, G$, may be taken as closely representing the actual area, $D\, K\, G\, F$. The reaction for both sides of the channel is given by,

$$\frac{\Delta w\, d^2}{2} + \frac{\Delta n}{3}(d + \Delta d)^3 = \frac{\Delta w\, d^2}{2} + \frac{\Delta n}{3}(d^3 + 3\, d^2\, \Delta d + 3\, d\, \Delta d^2 + \Delta d^3) \qquad \text{(4)}$$

Neglecting products and powers of Δ-quantities, Equation (4) becomes,

$$\frac{\Delta w\, d^2}{2} + \frac{\Delta n\, d^3}{3} \qquad \text{(5)}$$

(3) The retardation due to friction: This is very nearly equal to $\Delta x\, f\, (A + \frac{1}{2}\, \Delta A)$, but for short sections of any channels met with in practice, $\Delta x f$ will be very small, and $(A + \frac{1}{2}\, \Delta A)$ will be nearly equal to A. The retardation therefore, may be represented with sufficient accuracy by the term, $\Delta x\, f\, A$.

(4) The force due to gravity which may be represented by the term, $\Delta x\, s$: The momentum lost with the overflowing water is very nearly equal to,

$$\frac{\Delta Q\, (V + \frac{1}{2}\, \Delta V)}{g} \qquad \text{(6)}$$

in which, $V + \frac{1}{2}\, \Delta V$ is the average mean velocity in the length, Δx.

Equation (7) can now be written:

$$\frac{Q\, V}{g} + \frac{\Delta Q\, (V + \frac{1}{2}\, \Delta V)}{g} - \frac{(Q + \Delta Q)\,(V + \Delta V)}{g}$$

$$= \left\{\frac{(w + \Delta w)\,(d + \Delta d)^2}{2} + \frac{(n + \Delta n)\,(d + \Delta d)^3}{3}\right\} - \left(\frac{w\, d^2}{2} + \frac{n\, d^3}{3}\right)$$

$$- \frac{\Delta w\, d^2}{2} - \frac{\Delta n\, d^3}{3} - \Delta x\, (f - s)\, A \qquad \text{(7)}$$

Putting $V = \dfrac{Q}{A}$ and $V + \Delta V = \dfrac{Q + \Delta Q}{A + \Delta A}$ and rejecting products of Δ-quantities,

$$\frac{Q^2}{g\,A} + \frac{Q\,\Delta\,Q}{g\,A} - \frac{(Q + \Delta\,Q)^2}{g\,(A + \Delta\,A)} = \left\{ \frac{(w + \Delta\,w)\,(d^2 + 2\,d\,\Delta\,d)}{2} + \frac{(n + \Delta\,n)\,(d^3 + 3\,d^2\,\Delta\,d)}{3} \right\} - \frac{w\,d^2}{2} - \frac{n\,d^3}{3} - \frac{\Delta\,w\,d^2}{2}$$

$$- \frac{\Delta\,n\,d^3}{3} - \Delta\,x\,(f - s)\,(w\,d + n\,d^2) \;\ldots\ldots\ldots\ldots\ldots (8)$$

from which,

$$\frac{Q^2\,w\,\Delta\,d + Q^2\,\Delta\,w\,d + 2\,Q^2\,n\,d\,\Delta\,d + Q^2\,\Delta\,n\,d^2 - Q\,\Delta\,Q\,w\,d - Q\,\Delta\,Q\,n\,d^2}{g\left\{(w\,d + n\,d^2) + (w + 2\,n\,d)\,\Delta\,d + \Delta\,w\,d + \Delta\,n\,d^2\right\}(w\,d + n\,d^2)}$$

$$= (w\,d + n\,d^2)\,\Delta\,d - \Delta\,x\,(f - s)\,(w\,d + n\,d^2)$$

rejecting Δ-terms in the denominator, this reduces to,

$$\frac{\Delta\,d}{\Delta\,x} = \frac{(f - s) - \dfrac{Q}{g} \times \dfrac{1}{(w\,d + n\,d^2)^2} \times \dfrac{\Delta\,Q}{\Delta\,x} + \dfrac{Q^2}{g} \times \dfrac{d}{(w\,d + n\,d^2)^3} \times \dfrac{\Delta\,w}{\Delta\,x} + \dfrac{Q^2}{g} \times \dfrac{d^2}{(w\,d + n\,d^2)^3} \times \dfrac{\Delta\,n}{\Delta\,x}}{1 - \dfrac{Q^2}{g} \times \dfrac{w + 2\,n\,d}{(w\,d + n\,d^2)^3}} \;\ldots\ldots (9)$$

In Equation (9), if the channel is not acting as a spillway, then $\Delta\,Q$ vanishes and the second term of the numerator vanishes. If w be constant, the third term vanishes, and, if n be constant, the fourth term vanishes. The equation then reduces to:

$$\frac{\Delta\,d}{\Delta\,x} = \frac{f - s}{1 - \dfrac{Q^2}{g} \times \dfrac{w + 2\,n\,d}{(w\,d + n\,d^2)^3}} \;\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

For a rectangular channel, $n = 0$, and the equation further reduces to:

$$\frac{\Delta\,d}{\Delta\,x} = \frac{f - s}{1 - \dfrac{Q^2}{g} \times \dfrac{w}{w^3\,d^3}} = \frac{f - s}{1 - \dfrac{Q^2}{g\,A^2\,d}} = \frac{f - s}{1 - \dfrac{V^2}{g\,d}} \;\ldots\ldots\ldots (11)$$

Also, if the slope of the bed of the channel is just sufficient to overcome the resistance due to friction, then $f = s$, and $\Delta\,d$ vanishes, or, in other words, the depth becomes constant which is the condition of normal flow in a uniform channel.

Again, when,

$$\frac{Q^2}{g}\,\frac{w + 2\,n\,d}{(w\,d + n\,d^2)^3} = \frac{Q^2}{g\,A^2}\,\frac{w + 2\,n\,d}{w\,d + n\,d^2} = \frac{V^2}{g}\,\frac{w + 2\,n\,d}{w\,d + n\,d^2} = 1 \;\ldots\ldots (12)$$

that is, when,

$$\frac{w\,d + n\,d^2}{w + 2\,n\,d} = \frac{V^2}{g} = \text{twice the velocity head} \;\ldots\ldots\ldots\ldots (13)$$

the denominator vanishes, and the ratio, $\frac{\Delta d}{\Delta x}$, becomes infinite in the limit, or, in other words, the water surface becomes vertical. This can only occur when passing through the critical depth. For a rectangular channel, as n is then zero, the critical depth occurs where the depth is twice the velocity head, or,

$$d = \frac{V^2}{g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

For a channel, such as the spillway, into which water flows from the reservoir, the water enters at right angles to the axis of the channel and introduces no additional momentum corresponding to the loss of momentum which takes place when water flows outward from the channel.

The equation of flow for this condition is given by Equation (7) if the term, $\frac{\Delta Q\,(V + \frac{1}{2}\Delta V)}{g}$, is omitted, and a new equation corresponding to Equation (9) is then obtained:

$$\frac{\Delta d}{\Delta x} = \frac{(f - s) - \frac{2\,Q}{g}\,\frac{1}{(w\,d + n\,d^2)^2}\,\frac{\Delta Q}{\Delta x} + \frac{Q^2}{g}\,\frac{d}{(w\,d + n\,d^2)^3}\,\frac{\Delta w}{\Delta x} + \frac{Q^2}{g}\,\frac{d^2}{(w\,d + n\,d^2)^3}\,\frac{\Delta n}{\Delta x}}{1 - \frac{Q^2}{g}\,\frac{w + 2\,n\,d}{(w\,d + n\,d^2)^3}} \quad \ldots\ldots (15)$$

By writing y for d, and b for ΔQ, and omitting the friction term f, it can be shown that Equation (15) is another form of the equation,

$$\frac{d\,y}{d\,x} = \frac{V}{g}\,\frac{d\,V}{d\,x} + \frac{b\,V^2}{Q\,g} \quad \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

which is given by Julian Hinds, M. Am. Soc. C. E., in his paper on "Side Channel Spillways".*

Flume Experiments†

Equations (9) and (15) are only strictly correct when the average velocity is equal to the mean velocity, that is when,

$$\frac{1}{2}\,M\,V^2 = \frac{1}{2}\,\Sigma\,m\,v^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (17)$$

in which, for any elementary stream tube, m represents the mass passing the section per second, and v the velocity. This condition is never quite satisfied in any ordinary stream owing to the irregular distribution of velocity over the cross-section. In the case of a channel discharging a large quantity of water over the side, it was thought that the disturbance set up by eddies might be

* *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 881.

† The Liawenee experiments were carried out in November, 1922, and January, 1923. A full description of these experiments and the theory of flow in an overflowing channel are contained in an unpublished thesis presented by the writer to the University of Melbourne early in 1924.

TABLE 1.—COMPUTATION BY EQUATION (9) FOR WATER SURFACE CURVE IN LIAWENEE FLUME EXPERIMENT No. 19, SHOWN IN FIG. 1.

Distance from gate, in feet. (1)	Δx. (2)	d. (3)	w. (4)	Δw. (5)	n. (6)	Δn. (7)	Q. (8)	D. (9)	ΔQ. (10)	V. (11)	r. (12)	f. (13)	s. (14)	$\Delta x\,(f - s)$. (15)	$\frac{Q^2}{g} \cdot \frac{d}{(w\,d + n\,d^2)^3} \cdot \Delta w$. (16)	$\frac{Q^2}{g} \cdot \frac{d^2}{(w\,d + n\,d^2)^3} \cdot \Delta n$. (17)	$\frac{Q}{g} \cdot \frac{1}{(w\,d + n\,d^2)^2} \cdot \Delta Q$. (18)	Sum of Columns (15), (16), (17), and (18). (19)	$1 - \frac{Q^2}{g} \cdot \frac{w + 2\,n\,d}{(w\,d + n\,d^2)^3}$ (20)	Δd, Column (19) ÷ Column (20). (21)
30	..	6.20	5.67	.....	.667	....	125.0	....	....	2.06	2.96	0.00010	0.00303	−0.0293	......	....	......	−0.0293	0.969	−0.080
40	10	6.17	5.67	.....	.667	....	125.0	1.22	....	2.09	3.14	0.00010	0.00303	−0.0293	......	....	0.0491	−0.0784	0.971	−0.081
50	10	6.089	5.67	.....	.667	....	170.1	1.11	45.1	2.89	3.10	0.00018	0.00303	−0.0285	......	....	0.0597	0.0882	0.942	−0.094
60	10	5.995	5.67	.....	.667	....	209.1	0.99	39.0	3.63	3.05	0.00029	0.00303	−0.0274	......	....	0.0662	0.0936	0.907	−0.103
70	10	5.892	5.67	.....	.667	....	242.7	0.93	33.6	4.32	3.01	0.00042	0.00303	−0.0261	......	....	0.0732	0.0993	0.866	−0.115
80	10	5.777	5.67	.....	.667	....	273.2	0.88	30.5	4.73	2.95	0.00053	0.00303	−0.0250	......	....	0.0652	0.0902	0.836	−0.108
90	10	5.669	5.67	.....	.667	....	298.7	0.72	25.5	5.60	2.89	0.00073	0.00303	−0.0230	......	....	0.0650	0.0880	0.765	−0.115
100	10	5.554	5.67	.....	.667	....	318.5	0.63	19.8	6.13	2.81	0.00091	0.00303	−0.0212	......	....	0.0597	0.0809	0.710	−0.114
110	10	5.440	5.67	.....	.667	....	334.7	0.56	16.2	6.63	2.78	0.00110	0.00303	−0.0193	......	....	0.0598	0.0791	0.656	−0.120
120	10	5.320	5.67	.....	.667	....	349.3	0.54	14.5	7.12	2.72	0.00128	0.00303	−0.0175	......	....	0.0478	0.0653	0.594	−0.110
130	10	5.210	5.67	.....	.667	....	359.8	0.33	10.5	7.55	2.66	0.00146	0.00303	−0.0157	......	....	0.0284	0.0441	0.533	−0.083
140	10	5.127	5.67	.....	.667	....	365.5	0.26	5.7	7.85	2.61	0.00164	0.00303	−0.0139	......	....	0.0156	0.0295	0.486	−0.061
150	10	5.066	5.67	.....	.667	....	368.5	0.12	3.0	8.04	2.58	0.00172	0.00303	−0.0131	......	....	0.0057	0.0188	0.454	−0.042
	10								1.0											

TABLE 1.—(*Continued.*)

Distance from gate, in feet. (1)	Δx. (2)	d. (3)	w. (4)	Δw. (5)	n. (6)	Δn. (7)	Q. (8)	D. (9)	ΔQ. (10)	V. (11)	r. (12)	f. (13)	s. (14)	$\Delta x\,(f-s)$. (15)	$\frac{Q^2}{g}\cdot\frac{d}{(w\,d+n\,d^2)^3}\cdot\Delta w$. (16)	$\frac{Q^2}{g}\cdot\frac{d^2}{(w\,d+n\,d^2)^3}\cdot\Delta n$. (17)	$\frac{Q}{g}\cdot\frac{1}{(w\,d+n\,d^2)^2}\cdot\Delta Q$. (18)	Sum of Columns (15), (16), (17), and (18). (19)	$1-\frac{Q^2}{g}\cdot\frac{w+2\,n\,d}{(w\,d+n\,d^2)^3}$. (20)	Δd, Column (19) ÷ Column (20). (21)
160	10	5.024	5.67	.....	.667	....	369.5	0.06	0.3	8.15	2.56	0.00180	0.00303	—0.0123	......	....	0.0015	0.0138	0.434	—0.032
170	2	4.992	5.67	.....	.667	....	369.8	....	....	8.23	2.54	0.00200	0.00303	—0.00206	......	....	......	—0.00206	0.423	—0.005
172	1	4.987	5.67	0.278	.667	—0.0555	369.8	....	....	8.23	2.54	0.00200	0.00303	—0.00103	0.0652	—0.0650	......	—0.00083	0.424	—0.002
173	1	4.985	5.948	0.278	0.611	—0.0555	369.8	....	....	8.24	2.54	0.00198	0.00303	—0.00210	0.0655	—0.0652	......	—0.00075	0.43[illegible]	—0.002
174	1	4.983	6.226	0.278	0.555	—0.0555	369.8	....	....	8.24	2.54	0.00198	0.00303	—0.00105	0.0656	—0.0652	......	—0.00065	0.444	—0.001
175	2	4.982	6.50	0.56	0.499	—0.111	369.8	....	....	8.25	2.54	0.00198	0.00303	—0.00210	0.1320	—0.1304	......	—0.0005	0.458	—0.001
177	7	4.981	7.06	1.94	0.388	—0.388	369.8	....	....	8.24	2.52	0.00200	0.00303	—0.00721	0.4570	—0.4560	......	—0.0062	0.483	—0.013
184	5	4.968	9.00	2.00	0.000	0.000	369.8	....	....	8.27	2.36	0.00215	0.08000	—0.38925	0.4750	....	......	0.0858	0.573	0.150
189	5	5.118	11.00	3.50	0.000	....	369.8	....	....	6.68	2.60	0.00125	0.08000	—0.39375	0.4290	....	......	0.0358	0.735	0.048
194	5	5.116	14.50	3.00	0.000	....	369.8	....	....	4.93	3.02	0.00105	0.08000	—0.39475	0.1550	....	......	—0.2397	0.855	—0.280
199	5	4.886	17.50	1.50	0.000	....	369.8	....	....	4.32	3.13	0.00077	0.08000	—0.39615	0.0501	....	......	—0.3470	0.880	—0.394
204	15	4.492	19.00	9.00	0.000	....	369.8	....	....	4.32	3.05	0.00076	0.08000	—1.0885	0.2760	....	......	—0.9120	0.870	—1.048
219		3.835	28.00	.....	.....	....	369.8	....	....	3.88	....	....	....	....	......	....	......	....	.....	

sufficient to vitiate entirely the results obtained from Equation (9) and, therefore, it was considered necessary to make a large scale experiment before applying the equation to the design of new head-works.

Fortunately, the timber flume made it possible to carry out an experiment—known as Liawenee Flume Experiment No. 19—the results of which are shown in Fig. 1 and Table 1. The lip of the spillway was slightly irregular owing to warping of the timber, and the profile of it, shown by the heavier full line in Fig. 1, was obtained by spirit leveling. The observed water surface, shown by the thinner full line in Fig. 1, was also obtained by spirit leveling, the staff being held close to the wall of the channel on the side opposite the spillway and it is, therefore, probably slightly higher than the mean level of the water surface.

Owing to the centrifugal effect in the curved converging inlet and the fact that the velocity of approach in the river above the weir slightly heaps up the water at the point marked, H, in Fig. 1, the water level on the two sides of the channel differs considerably. The points lettered on the longitudinal section correspond with those on the plan. The grade of the floor in the converging inlet had not been accurately determined and, for that reason, it is shown dotted in Fig. 1. The walls of the inlet are cement, but the floor is rough and the value of Kutter's n has been taken as 0.020. In the computations in Table 1, Kutter's n for the timber flume was taken as 0.015, although it was later found to be 0.017, but the effect of the friction term is so small that the level of the computed water surface is not appreciably affected thereby.

The flow in the flume was determined by current meter at a permanent gauging station on a uniform section of the canal a few hundred feet down stream from the outlet of the concrete flume. In this experiment, the total flow entering the timber flume was 360 sec-ft., of which quantity, 235 sec-ft. were discharged over the spillway, the remaining 125 sec-ft. passing under the head-gate, the lower edge of which was 1.37 ft. above the floor of the flume. It should be noted that spilling over the edge of the flume commenced 40 ft. up stream from the gate and ceased at 172 ft. and, therefore, was confined to a straight length of flume of normal cross-section, having a bed width of 5 ft. 8 in. and side slopes of $\frac{2}{3}$ horizontal to 1 vertical.

The water surface, shown by the dotted line in Fig. 1, was computed by Equation (9), the calculations being presented in Table 1. In order to find values of ΔQ for use in Table 1, it was necessary to determine the value of the coefficient for the spillway lip, a detailed cross-section of which is shown in Fig. 1. The mean value of the head, D, on the spillway was determined for each 4-ft. length, and the value of the coefficient, c, was found to be 3.57 from Equation (18):

$$\Sigma \Delta Q = c \, \Sigma (\Delta x \, D^{\frac{3}{2}}) = c \, \Sigma (4 \, D^{\frac{3}{2}}) = 235 \qquad (18)$$

In Equation (9), the expression, $(w\,d + n\,d^2)$, represents the complete trapezoid, $L\,R\,O\,P$ (see cross-section, Fig. 1), but, in the computation in Table 1, the area, $M\,R\,O\,P\,N$, was utilized as more nearly representing the

actual cross-section of the water area. This procedure resulted in more labor in calculation than would have been involved if the complete trapezoidal area had been adopted and as the maximum area of the triangle, $L\,M\,N$, is only 0.006 of the area, $L\,R\,O\,P$, no appreciable error would have resulted if the area, $L\,R\,O\,P$, had been adopted instead of the area, $M\,R\,O\,P\,N$.

The actual water-surface level at a point 30 ft. up stream from the gate was adopted as a starting point from which to obtain the computed water-surface curve (dotted line, Fig. 1) which, in general, agrees closely with the actual water surface. From the head-gate to Station 40, the flume is curved and the centrifugal effect here would be to lower the actual water surface along the side of the flume opposite the spillway and this is probably the explanation of the divergence between the actual and computed water surfaces between Stations 30 and 60. The critical depth for the flume for a flow of 360 sec-ft. is 4.22 ft. and as the waves, which occur at the up-stream end of the flume, do not extend down to the critical depth, they are not hydraulic jumps. The water issues from the curved inlet at a velocity of 8.27 ft. per sec. and, as the upper portion of the walls of the timber transition flare outward from the end of the concrete inlet, the rapidly moving stream cannot immediately accommodate itself to the changed direction of the walls, and, consequently, a partial vacuum occurs against the upper portion of the walls, and is marked by a small area of turbulence which can be seen at the extreme left in Fig. 5. Equation (9) is based on the assumption that the water exerts a pressure against the walls at all points equal to the full static pressure due to the depth, and the waves are due to the fact that this condition does not hold just at the entrance to the transition. Up stream from the throat the computed water surface agrees fairly well with the mean of the observed water surfaces in the curved inlet. The computed total discharge of 370 sec-ft. (see Table 1) agrees well with the measured discharge of 360 sec-ft.

Proposed Design for New Head-Works for Ouse-Great Lake Canal

As the foregoing experiment showed that Equation (9) could be satisfactorily applied in practice on a comparatively large scale, it was decided to make a new design for the head-works of the canal based on the theory just developed. A diagrammatic longitudinal section of the proposed head-works is shown in Fig. 6. The existing curved inlet, shown in Fig. 1, from Station 184 up stream will be retained. At Station 184 a square-edged concrete baffle-wall will be constructed above the throat of the inlet and will extend across to the wall of the gorge so that water can enter the flume only by means of the orifice, 5 ft. high by 9 ft. wide, under the baffle. No gate is required at this point, but provision for stop-planks will be made. A second orifice, 5 ft. high, will be formed by a square-edged concrete head-wall in the position occupied by the existing head-gate, and a sector gate having the full width of the concrete flume will be provided for closing this orifice when it is desired to prevent water entering the canal.

Between Stations 0 and 184, the existing timber flume will be replaced by a rectangular concrete flume, 9 ft. wide, designed to act as a spillway between Stations 30 and 152. It will be noted that, between Stations 10 and 174, the

floor of the flume is lowered about 5 ft. below the normal grade line of 1 in 330 to form a deep pool throughout the spillway portion of the flume. In the actual design the sharp changes of grade shown in Fig. 6 at Stations 10, 30, 152, and 174 will be rounded, but the hydraulic computations are simplified by considering them as being sharp and this assumption has no appreciable effect on the results.

Before investigating the behavior of the proposed head-works in detail, it is necessary to decide on suitable values for the coefficient of roughness of the concrete flume and the coefficients of discharge of the spillway and the orifice at the inlet and head-gate.

Coefficient of Roughness of Flume.—Gaugings taken in the existing concrete flume show that the value of Kutter's n varies from 0.014 in summer to 0.015 in winter, owing to the fact that a growth appears on the walls and floor during the high-water period when the flume is continuously filled. This growth dies again if the flow ceases frequently. The value of 0.015 was adopted for Kutter's n, but the shape of the water-surface curves shown in Fig. 6 would be very slightly affected by a considerable error in the value of n.

Coefficient of Discharge for Spillway.—A cross-section of the crest of the proposed spillway is shown in Fig. 6. For a triangular weir, sharp crest, up-stream face vertical, top sloping 2 horizontal to 1 vertical, the value of c in Equation (1):

$$\Delta Q = c \, \Delta x \, D^{\frac{3}{2}}$$

for a head of 1 ft. on the crest is given by King* as 3.50. Bazin's experiments on wide crested weirs showed an increase of 9% in the value of c when the up-stream edge was rounded to a radius of 4 in. Assuming the same increase for the spillway, the value of c would become 3.82. A value of 3.80 was adopted for the spillway.

Coefficient of Discharge of Orifices.—At both the inlet and head-gate orifices, the floor and the two walls of the flume are continuous so that contraction is entirely suppressed on these three sides, but the top of the orifice is sharp-edged and the velocity of approach is usually considerable. No published experiments dealing with orifices of this nature could be found and, therefore, some experiments (referred to as the "Baffle Experiments") were carried out in January, 1923, by placing a timber baffle in a straight portion of the existing concrete flume. The data obtained from Experiments B2-6 and the profiles of the water surface are shown in Table 2 and Fig. 7 which is self-explanatory. The bottom of the baffle in all experiments was sharp-edged and was located at Distance 0; in Experiment 3 the baffle was inclined 45° up stream. The depth, d_1, was measured 5 ft. up stream from the baffle. The floor of the flume had a grade of 1 in 330. In Experiment B-1 the water backed up and overflowed the wall of the flume, therefore, the results have been discarded. The water-surface profiles were obtained by measuring downward, with a rod graduated in inches, from points on the wall of the flume, the elevations of which had been determined with an engineer's level. As the result of these experiments, 0.58 was adopted as the value of c for the inlet and head-gate.

* "Handbook of Hydraulics."

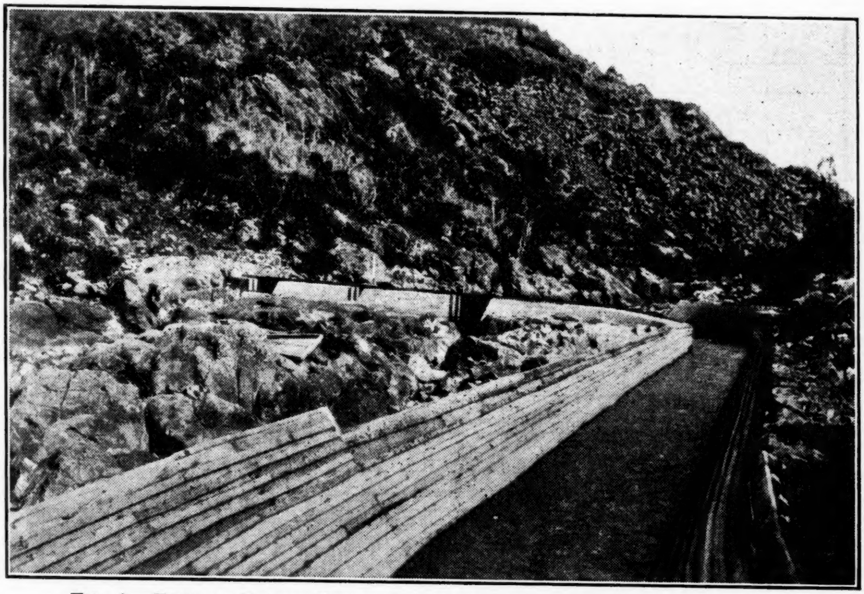


FIG. 4.—VIEW OF TIMBER FLUME, LOOKING UP STREAM FROM THE HEAD-GATE.

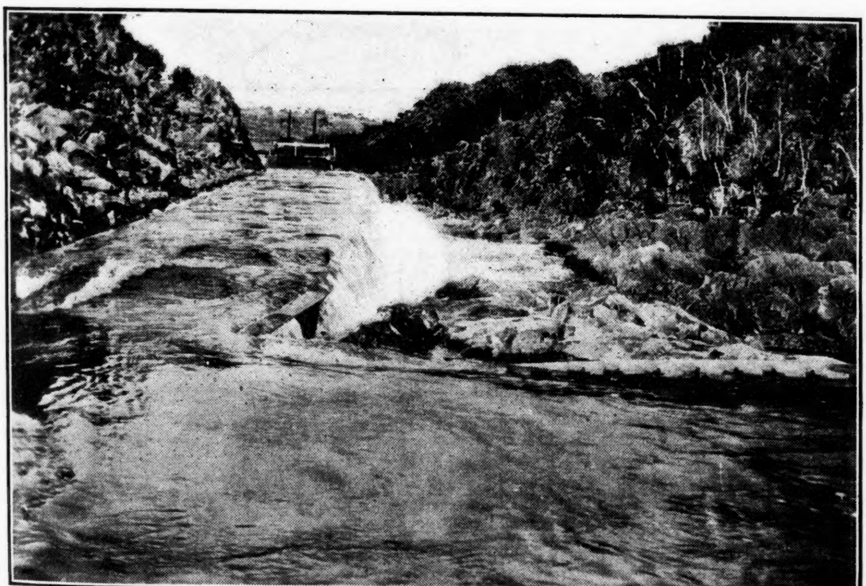


FIG. 5.—VIEW SHOWING ENTIRE FLOW OF OUSE RIVER DISCHARGING OVER SIDE OF TIMBER FLUME.

Pa

Elevation in Feet
3454
3453
3452
3451
3450
3449
3448
3447
3446
3445
3444
3443
3442
3441
3440

Fi

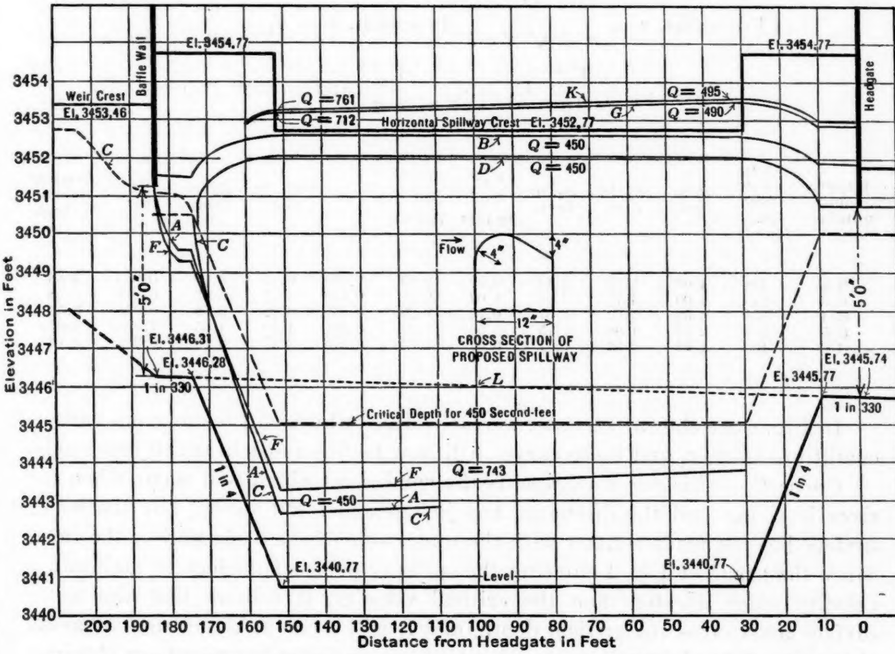


FIG. 6.—PROPOSED DESIGN FOR HEAD-WORKS. DIAGRAMMATIC SECTION ON CENTER LINE.

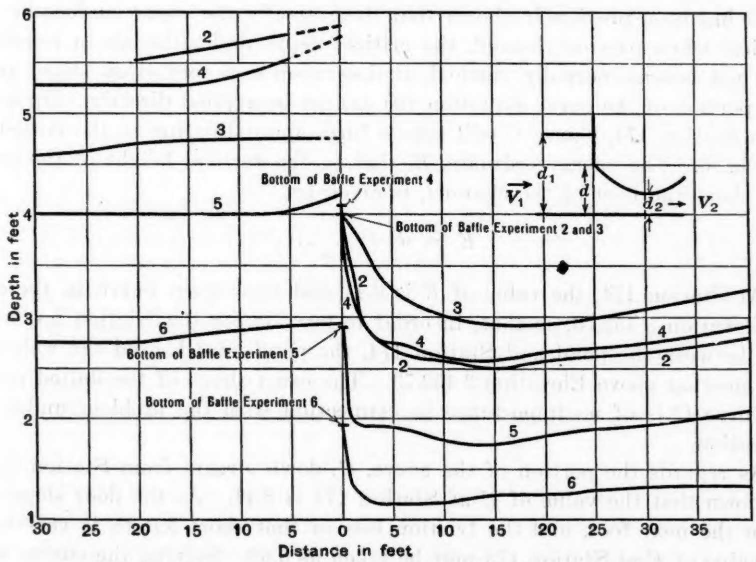


FIG. 7.—PROFILES OF WATER SURFACE, BAFFLE EXPERIMENTS.

TABLE 2.—BAFFLE EXPERIMENTS, JANUARY 8 TO 11, 1923.

$$\left(\text{Formulas, } c = \frac{Q}{w\,d\,\sqrt{2\,g\,h}}, \text{ in which, } h = d_1 - \frac{d}{2} + \frac{V_1^2}{2\,g};\right.$$
$$\left. E_1 = d_1 + \frac{V_1^2}{2\,g};\ E_2 = d_2 + \frac{V_2^2}{2\,g};\ \text{and } w = 9 \text{ ft.}\right)$$

Experiment number.	Q, in cubic feet per second.	d, in feet.	d_1, in feet.	d_2, in feet.	$\frac{V_1^2}{2\,g}$, in feet.	$\frac{V_2^2}{2\,g}$, in feet.	c.	E_1, in feet.	E_2, in feet.	$\frac{E_2}{E_1}$.	Critical depth, in feet.
B-2	346	4.00	5.76	2.50	0.69	3.69	0.569	6.45	6.19	0.960	3.58
B-3	346	4.00	4.75	2.92	1.02	2.71	0.618	5.77	5.63	0.975	3.58
B-4	336	4.08	5.52	2.57	0.72	3.29	0.557	6.24	5.86	0.938	3.53
B-5	215	2.92	4.00	1.75	0.56	2.90	0.581	4.56	4.65	1.020	2.61
B-6	120	1.92	2.92	1.17	0.33	2.02	0.575	3.25	3.19	0.982	1.78

In Fig. 6 are shown the water surface curves which will result from various conditions of flow, and these curves will now be discussed in detail separately.

Curve C.—This curve represents the conditions which will occur when the river is rising and the discharge has just reached 450 sec-ft., but the water surface has not made contact with the under side of the baffle-wall at the inlet. Since the grade of 1 in 4 entering the spillway pool is sufficient to maintain a velocity much greater than the critical velocity, it follows that the water surface must cross the critical depth, and it will do so at the change of grade at Station 174. A known point on a back-water curve has now been obtained and, in most cases, the form of the curve could be computed, in either the up-stream or down-stream direction, by Equation (9).

It has been previously shown that theoretically the water surface becomes vertical when passing through the critical depth and although in practice it does not become actually vertical, it does assume a very steep slope, and to this portion of the curve Equation (9) cannot be applied directly. Up stream from Station 174, Curve C will take a form approximating to the dotted line on Fig. 6. The energy ordinate, E, that is, the vertical height of the energy line above the floor of the channel, is given by,

$$E = d + \frac{V^2}{2\,g} \qquad (19)$$

At Station 174, the value of E is 6.41 and the energy curve is, therefore, at Elevation 3 452.73, so that, in order to provide for the friction loss occurring between the pond and Station 174, the pond level behind the weir must be somewhat above Elevation 3 452.73. The exact shape of the dotted portion of Curve C is of no importance in connection with the problem under consideration.

As regards the portion of the curve, C, down stream from Station 174, it is known that the value of E at Station 174 is 6.41. As the floor slopes 0.25 ft. in the next foot, and the friction loss in that short length is very small, the value of E at Station 173 may be taken as 6.66. Solving the energy equation, the depth at Station 173 is found to be 3.54 ft. From Station 173 down

stream, Curve C has been computed by Equation (9) in a manner exactly similar to that used in computing the water surface in Experiment No. 19 (Fig. 1), as shown in detail in Table 1, except that, for Curve C, ΔQ, ΔW, and Δn are all zero, and the calculation is thereby simplified.

Curve D.—The normal depth for 450 sec-ft. in the concrete flume being 5.0 ft., therefore, if the water surface is not touching the under side of the head-gate, a depth of 5 ft. at Station 0 may be adopted as a starting point from which Curve D can be computed up stream by Equation (9).

In applying Equation (9) to those portions of the curve between Stations 10 and 30 and Stations 152 and 172.5, where the slope of the water surface is changing rapidly, it is necessary to take extremely small steps in making the calculations, but, owing to the great depth, the energy lost in friction in each of these portions does not exceed about 0.01 ft., and, therefore, no appreciable error is introduced by considering the energy line to be at the same elevation throughout these portions of the flume. The value of d can then be determined by solving the energy equation,

$$E = d + \frac{V^2}{2\,g} = d + \frac{Q^2}{2\,g\,w^2\,d^2} \qquad (20)$$

This method of dealing with the steeper portions of the curves avoids the great amount of labor involved in applying Equation (9) and has been adopted in determining such portions of all curves shown in Fig. 6. Between Stations 30 and 152, Curve D has been computed by Equation (9).

Curves C and D.—The water entering under the baffle cannot continue to follow Curve C indefinitely as the bottom of the flume is level, and energy is being absorbed by friction. In order to pass under the head-gate, the water must change by some means from Curve C to Curve D and, as this involves crossing the critical depth, the change can only take place through an hydraulic jump. An hydraulic jump between any two curves can only take place at a point where the value of the momentum,

$$M = \frac{A\,d}{2} + \frac{Q\,V}{g} \qquad (21)$$

is the same for both curves.

The curves in Fig. 8 show the value of M at various points on the correspondingly lettered curves in Fig. 6. Referring to Curves C and D in Fig. 8, it is seen that they intersect at Station 170 and, therefore, the water surface must jump from Curve C to Curve D (Fig. 6) at that point.

Curve A.—This curve represents the conditions which will occur with a falling stage of the river when the flow is 450 sec-ft. and the water is in contact with the under side of the baffle. From the formula,

$$Q = A\,c\,\sqrt{2\,g\,h} \qquad (22)$$

it is found that the head, h, over the center of gravity of the orifice is 4.65 ft. The crest of the weir, therefore, must be 7.15 ft. above the bottom of the orifice, that is, at Elevation 3 453.46, in order that no water shall be wasted over the weir until the flume is receiving 450 sec-ft.

The value of E on the up-stream side of the baffle is 7.15 and Baffle Experiments No. B 2-4 (Table 2 and Fig. 7) show that a loss of about 4% of the energy occurs in passing the baffle. Therefore, on the down-stream side of the baffle, the value of E will be about 6.86, from which it is found that the depth at about 6 ft. on the down-stream side of the baffle will be 3.33 ft. From Station 178 down stream, Curve A has been computed by Equation (9) and merges into Curve C.

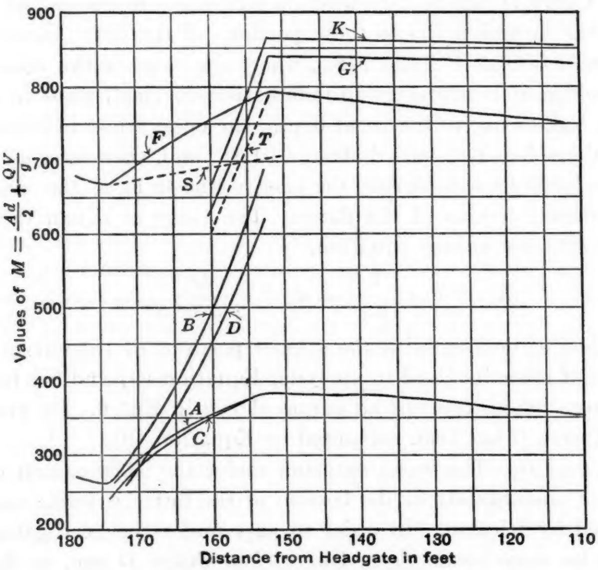


FIG. 8.—MOMENTUM CURVES.

Curve B.—This curve represents the conditions occurring with a falling stage of the river when the flow is 450 sec-ft. and the water touches the underside of the head-gate. From the formula, $Q = A\ c\ \sqrt{2\ g\ h}$,

$$h = d + \frac{V^2}{2\ g} - 2.5 = 6.92 \text{ ft.}$$

from which, for the up-stream side of the head-gate, d, by trial, equals 8.64 ft.

Curve B is then computed from the head-gate up stream in exactly the same manner as has been described in the case of Curve D. Curve B fixes the lowest level at which the spillway can be placed without losing water when the total flow in the river does not exceed the normal canal capacity of 450 sec-ft. To provide some margin for error in the calculations, the spillway has been placed slightly above Curve B, being horizontal and exactly 12.0 ft. above the floor of the channel.

Curves A and B.—The water, entering under the baffle and flowing along Curve A, must jump to Curve B in order to pass the head-gate. Reference to Fig. 8 shows that the momentum Curves A and B intersect at Station 173.5 and at this point the jump must take place.

Curve F.—This curve represents the surface curve that will be assumed by the water passing under the baffle during the maximum flood which is estimated to rise to a height of 8 ft. above the crest of the weir, that is, to a height of 15.15 ft. above the bottom of the orifice. Therefore, $h = 12.65$ ft., and,

$$Q = A\ c\ \sqrt{2\ g\ h} = 743 \text{ sec-ft.}$$

As the velocity head at the up-stream side of the baffle is negligible, the energy ordinate, E, at this point is equal to the depth, d_1, that is, 15.15 ft. Allowing 4% loss of energy in passing the baffle the energy ordinate on the down-stream side, E_2, becomes 14.55 ft., from which $d_2 = 3.04$ ft. is obtained by trial. This depth will occur about 6 ft. down stream from the baffle, that is, at Station 178. From Station 178 down stream, Curve F has been computed by Equation (9).

The critical depth, d_c, for 743 sec-ft. is given by,

$$d_c = \sqrt[3]{\frac{Q^2}{g\ w^2}} = 5.97 \text{ ft.} \qquad (23)$$

and it is obvious that the water cannot pass through the head-gate without first jumping across the critical depth to some curve, such as G or K, and while flowing along such a curve a large quantity of water will be discharged over the spillway, although the flow under the head-gate will also be somewhat in excess of the normal canal capacity of 450 sec-ft.

For the flow to take place along any curve, such as G or K, two conditions must be satisfied: (1) That the value of M on the given curve, at the point where the jump takes place, must be equal to the value of M for Curve F at the corresponding point; and (2) the sum of the discharges over the spillway and under the head-gate must be 743 sec-ft. The curve which will satisfy these conditions can be found by trial as explained subsequently.

Curve G.—Let it be assumed that the quantity of water passing under the head-gate is 490 sec-ft., then, in the same manner as has been explained in regard to Curve B, it is found that the depth, d_1, against the up-stream side of the gate is 7.07 ft. From Station 0 to Station 30 and from Station 152 to Station 160, Curve G has been computed by the energy equation as previously explained for Curve D. Throughout the length of the spillway, that is, between Stations 30 and 152, Curve G has been computed by Equation (9), and the computation is shown in Table 3.

Referring to the momentum curves in Fig. 8, it will be seen that Curves F and G intersect at Station 154, and, therefore, at this point the water can jump from Curve F to Curve G. On referring to Table 3, it is noted that the value of Q increases from 490 sec-ft. at the down-stream end of the spillway (Station 30) to 712 sec-ft. at the up-stream end (Station 152), but this latter value should be 743 sec-ft. in order to satisfy Condition (2) as stated. The water surface, therefore, must take up some position above Curve G.

Curve K.—Assuming that the quantity of water passing under the head-gate is 495 sec-ft., it is found that the depth, d_1, against the head-gate is 7.20 ft., and from this point Curve K is computed in exactly the same manner as Curve G. In Fig. 8, it is seen that the momentum curves, F and K, intersect

TABLE 3.—COMPUTATION OF CURVES G AND K, (FIG. 6).

	Distance from gate, in feet.	Δx.	d.	D.	Q.	ΔQ.	V.	r.	f.	s.	$\Delta x (f - s)$.	$\frac{Q}{g} \cdot \frac{1}{(w d + n d^2)^2} \cdot \Delta Q$.	Sum of Columns (11) and (12).	$1 - \frac{Q^2}{g} \cdot \frac{w + 2 n d}{(w d + n d^2)^3}$.	Δd Column (13) ÷ Column (14).
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
Curve G	30	20	12.73	0.73	490.0	47.4	4.28	3.32	0.000371	0.000	0.0074	0.0553	−0.0479	0.9552	−0.050
	50	20	12.68	0.68	537.4	42.6	4.71	3.32	0.000435	0 000	0.0087	0.0548	−0.0461	0.9403	−0.049
	70	20	12.631	0.631	580.0	38.1	5.10	3.31	0.000527	0.000	0.0105	0.0534	−0.0429	0.9355	−0.046
	90	20	12.585	0.585	618.1	34.0	5.45	3.32	0.000588	0.000	0.0117	0.0511	−0.0394	0.9262	−0.043
	110	20	12.542	0.542	652.1	30.3	5.78	3.31	0.000667	0.000	0.0133	0.0485	−0.0352	0.9167	−0.038
	130	22	12.504	0.504	682.4	29.9	6.06	3.31	0.000715	0.000	0.0157	0.0504	−0.0347	0.9080	−0.038
	152		12.466	......	712.3		6.35	....	....	......	......	......	....	......	....
Curve K	30	10	12.84	0.84	495.0	29.2	4.28	3.33	0.000412	0.000	0.0041	0.0338	−0.0297	0.9568	−0.031
	40	20	12.809	0.809	524.2	55.2	4.54	3.33	0.000400	0.000	0.0080	0.0678	−0.0598	0.9496	−0.063
	60	20	12.746	0.746	579.4	48.9	5.06	3.32	0.000500	0.000	0.0100	0.0669	−0.0569	0.9374	−0.061
	80	20	12.685	0.685	628.3	43.1	5.50	3.32	0.000606	0.000	0.0121	0.0648	−0.0527	0.9173	−0.057
	100	30	12.628	0.628	671.4	56.7	5.91	3.32	0.000713	0.000	0.0214	0.0923	−0.0709	0.9134	−0.078
	130	22	12.550	0.550	728.1	32.6	6.45	3.31	0.000832	0.000	0.0183	0.0582	−0.0399	0.8964	−0.044
	152		12.506	......	760.7		6.76	....	....	......	......	......	....	......	....

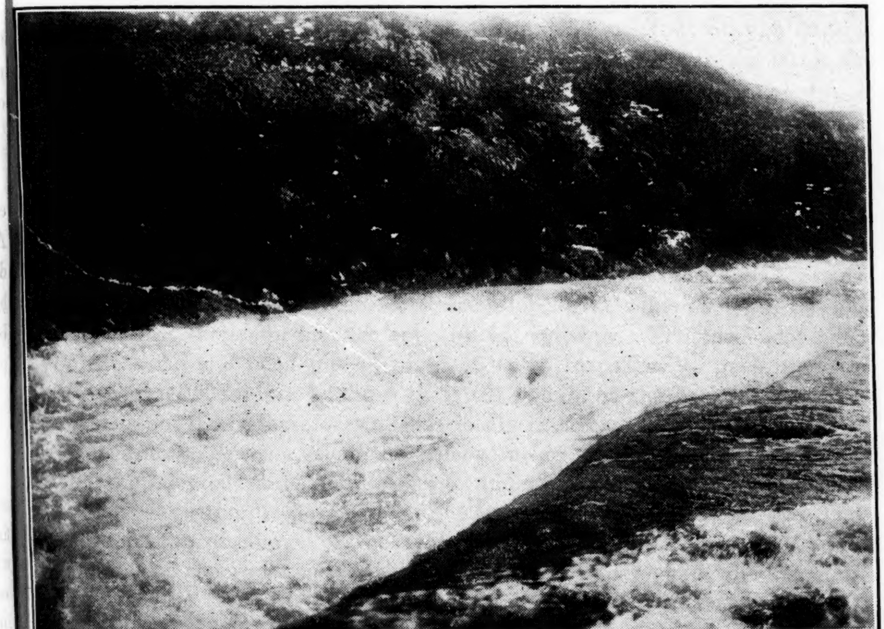


FIG. 9.—VIEW OF TIMBER FLUME, UP STREAM FROM THE HEAD-GATE, SHOWING HYDRAULIC JUMP.

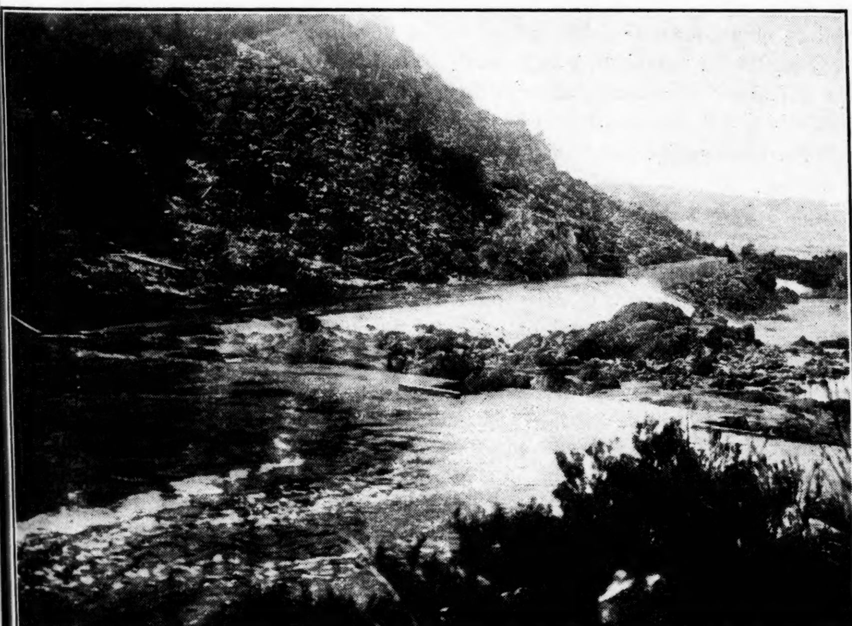


FIG. 10.—VIEW OF TIMBER FLUME UNDER NORMAL CONDITIONS.

at Station 155, which is the only possible position for the jump. Table 3, however, shows that the value of Q increases from 495 sec-ft. at Station 30 to 761 sec-ft. at Station 152, but, as in order to satisfy the conditions the latter value should be 743 sec-ft., it follows that the water surface must take up some position below Curve K and, therefore, it must occupy some position between Curves G and K.

It is necessary to consider to what extent the degree of regulation will be affected by errors in the adopted values for Kutter's n and the coefficients of discharge of the baffle and head-gate and of the spillway. Table 3 shows that the friction term in the numerator of Equation (9) is very small compared with the term in ΔQ and, therefore, variation in the value of n will not appreciably affect the water surface opposite the spillway. The coefficient adopted for the baffle and head-gate is based on actual experiments made under conditions exactly similar to those that will obtain at the baffle and head-gate, and it is not likely therefore to be seriously in error.

Assuming, however, that the true value of the coefficient is 5% greater than the adopted value, the discharge under the baffle would be increased by 37 sec-ft. and the discharge under the head-gate by about 24 sec-ft. The increase in the discharge of the spillway would be only about 13 sec-ft. and the level of the water surface would be very slightly changed. An increase of 5% in the value of the coefficient, therefore, would increase the excess in the canal to about 15% of the normal flow. A small decrease in the value of the coefficient, of course, will improve the regulation, but will raise the level of Curve B to some extent, for which reason the spillway crest has been placed a little higher than Curve B.

The coefficient for the spillway is the factor which is most likely to be in error, but Curves G and K (Table 3) show that a variation of 44 sec-ft., or 20% of the total discharge over the spillway, only causes a variation of 5 sec-ft. in the discharge under the head-gate, so that the value of the coefficient for the spillway may be considerably in error without appreciably affecting the regulation of the canal.

The proposed design for the head-works, therefore, will probably prevent the normal capacity of the canal being exceeded by more than 10% and will certainly prevent it being exceeded by more than 15% which is equivalent to 70 sec-ft. Since the canal is provided with spillways where it crosses natural drainage channels and can be operated for a reasonable period under an overload of 100 sec-ft., the proposed design which involves no moving parts and is, therefore, "fool-proof" and frost proof, will provide satisfactory regulation.

If the floor of the spillway portion of the channel were not lowered, but were constructed to the dotted line, L, in Fig. 6, the momentum, M_L, of the water leaving the baffle orifice at the lower stage, would exceed the momentum, M_U, for the upper stage, which would be the normal depth. If a jump is to occur, the value of M_L must be decreased until it equals the value of M_U. As friction would be the only force tending to decrease the value of M_L, the high velocity stage might persist to a point beyond the head-gate. Therefore, in order

to make the spillway effective, some means must be adopted to ensure that a jump will occur at some definite point close to the baffle. Similar conditions were met in the experiments of the Miami Conservancy District* in the design of outlets from its flood-detention reservoirs. In that case the reaction of a submerged obstruction was used to decrease the value of M_L, but in order to avoid raising the water level too high in passing over the obstruction, the obstruction was placed below the normal floor of the channel and the water led to it down an incline which had the effect of first raising the value of M_L.

In the proposed head-works design shown in Fig. 6, a definite location for the jump is secured in a different manner. The function of the deep pool is to make the value of M_U greater than that of M_L. Instead of decreasing M_L, it is necessary, therefore, either to increase M_L or to decrease M_U, or to do both. The curves in Fig. 8 show clearly that the steep grade of the floor in Fig. 6 does both increase M_L and decrease M_U, causing the curves to intersect at a decided angle and thereby definitely locating the jump within the limits of the steep grade.

The view, Fig. 9, looking up stream from the head-gate during a moderate flood in the river, shows a good example of an hydraulic jump occurring at about the center of the timber flume. The jump is due to the fact that the head-gate was partly closed, thus backing up the water in the flume. In comparison, Fig. 10 shows the flume under more normal conditions.

For the particular head-works under discussion, the very narrow space available at the side of the gorge renders the method of obtaining regulation by lowering the floor very suitable. It is not the only possible method, however, and similar results could be achieved by widening the channel behind the baffle at the inlet and contracting it again at the head-gate. Thus, if the width of the floor be increased from 9 ft. at Station 174 (Fig. 6) to 40 ft. at Station 152, the dotted curve, S, in Fig. 8 shows the variation of M_L for a discharge of 743 sec-ft. The dotted curve, T, shows the approximate value of M_U, assuming a depth of 1 ft. of water over the spillway at Station 152. Although no incline is used, the intersection of Curves S and T (Fig. 8) shows that the jump will be just as definitely located in the transition in this instance as it is for the design shown in Fig. 6 and many cases might arise in practice where it is preferable to widen the channel rather than deepen it.

For the Ouse-Great Lake Canal, regulation of the flow within 15% of the normal is sufficient, but if closer regulation were desired it could be obtained by installing a second baffle, stilling pool, and spillway immediately down stream from Station 0 (Fig. 6) and moving the head-gate to the down-stream end of this second pool. If necessary, a series of stilling pools and spillways, separated by orifices, might be used. The problem of regulating the flow in a diversion channel taking water from a river subject to rapid and considerable changes of level is one which must frequently be encountered, and where some type of automatic gate is not suitable, the principles of the design illustrated in Fig. 6 may find a practical application.

* Miami Conservancy District, Technical Reports, Pt. 3.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

A GRAPHIC METHOD FOR DETERMINING THE STRESSES IN CIRCULAR ARCHES UNDER NORMAL LOADS BY THE CAIN FORMULAS

By Frederick Hall Fowler,* M. Am. Soc. C. E.

Synopsis

This paper presents a series of curves for determining the stresses, in pounds per square inch, at the extrados and intrados of crown and abutments of circular arches under normal loads.

The curves fall in three groups based on three separate sets of formulas developed by William Cain, M. Am. Soc. C. E.:

Group I.—Arches with "fixed ends" (neglecting shear). (Figs. 3, 4, 5, and 6.)

Group II.—Arches with "hinged ends" (neglecting shear). (Figs. 7, 8, and 9.)

Group III.—Arches with "fixed ends" (including influence of shear). (Figs. 10(*a*), 10(*b*), 11(*a*), 11(*b*), 12(*a*), 12(*b*), 13(*a*), and 13(*b*).)

The curves represent the stresses due to a full water load corresponding to a 10-ft. head; but, by the methods outlined, they may be used for any head. They afford a quick and easy method of applying a very complicated and rigorous method of analysis.

The problem of determining with reasonable accuracy the stresses in arch dams has commanded an important place in engineering literature, particularly during the past six years. In the *Transactions* and *Proceedings* of the Society alone, from 1919-20 to date, there have been seven papers† devoted

* Cons. Civ. Engr., San Francisco, Calif.

† "Improving Arch Action in Arch Dams," by L. R. Jorgensen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 316; "Arched Dams," by B. A. Smith, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2027; "Gravity and Arch Action in Curved Dams," by Fred A. Noetzli, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1; "The Circular Arch Under Normal Loads," by William Cain, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233; "The Relation Between Deflections and Stresses in Arch Dams," by F. A. Noetzli, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 284; "Stresses in Thick Arches of Dams," by B. F. Jakobsen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 475, containing a discussion and solution by the principle of least work, by William Cain, M. Am. Soc. C. E., p. 522; and "Experimental Deformation of a Cylindrical Arched Dam," by B. A. Smith, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., October, 1926, Papers and Discussions, p. 1596.

exclusively or in large part to the study of stresses in the ordinary arch dam, and three others* devoted to the special problems of the multiple arch.

These papers propose formulas that range from rough approximations to the most rigid mathematical determinations. Two valuable contributions of the group were by Professor Cain. In the first of these, "The Circular Arch Under Normal Loads", the author developed, by refined mathematical methods, formulas for computing moments and thrusts, and the resulting stresses in arches with "fixed" and "hinged" ends.

Fig. 1 taken from this paper is supposed to represent a horizontal circular arch, 1 ft. thick, perpendicular to the plane of the paper.

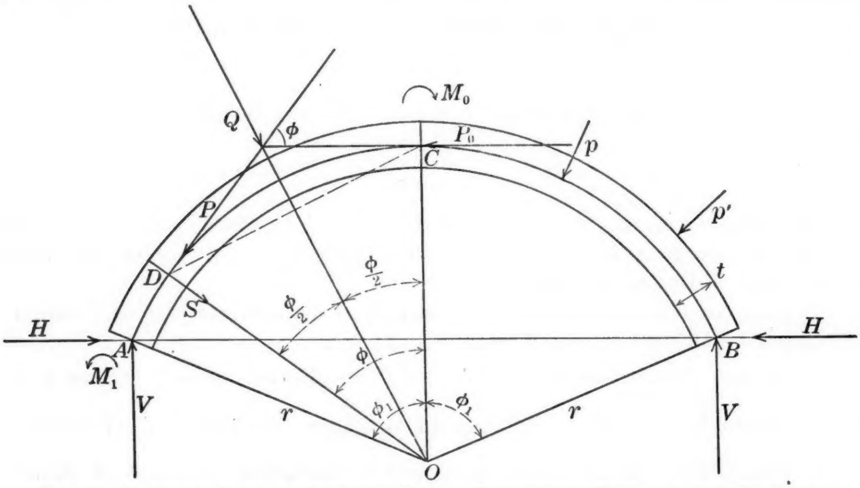


FIG. 1.—CIRCULAR ARCH OF UNIFORM RADIAL THICKNESS, FIXED AT THE ENDS AND SUBJECTED TO A UNIFORM NORMAL, RADIAL PRESSURE.

The notation used† (omitting symbols that did not enter into the formulas for moments and thrusts), was as follows:

Let t = uniform radial thickness of arch, in feet;

r = radius of center line of arch, in feet;

r' = radius of extrados, in feet;

p' = normal radial pressure, in pounds per square foot, on extrados;

p = normal pressure, in pounds per square foot, on center line $= \frac{p'\,r'}{r}$;

ϕ = angle with radius of crown for any point, D;

ϕ_1 = half central angle, $A\,O\,B$;

s = length of arc, $C\,D = r\,\phi$, $\therefore\ ds = r\,d\,\phi$;

* "Stresses in Multiple-Arch Dams," by B. F. Jakobsen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 276; "Improved Type of Multiple-Arch Dam," by Fred A. Noetzli, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 342; and, "Multiple-Arch Dam at Gem Lake on Rush Creek, California," by Fred O. Dolson and Walter L. Huber, Members, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 713.

† *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 235.

M_0 = moment at crown, taken positive clockwise;
P_0 = thrust at crown;
M and P are, respectively, the moment and tangential component of the thrust at D (r, ϕ).

With this notation, Professor Cain proceeded to develop a series of formulas, among which were the following for moments and thrusts at crown and abutment:

Arch fixed at ends:

$$P_0 = \text{thrust at crown} = p\,r - \frac{p\,r}{D}\,2\,\phi_1 \sin\phi_1\,\frac{k^2}{r^2} \quad \ldots\ldots\ldots\ldots\ldots (10)^*$$

$$M_0 = \text{moment at crown} = -\,(p\,r - P_0)\,r\left(1 - \frac{\sin\phi_1}{\phi_1}\right) \quad \ldots\ldots\ldots (12)^*$$

$$P_1 = \text{thrust at abutments} = p\,r - (p\,r - P_0)\cos\phi_1 \quad \ldots\ldots\ldots\ldots (4)^\dagger$$

$$M_1 = \text{moment at abutments} = r\,(p\,r - P_0)\left(\frac{\sin\phi_1}{\phi_1} - \cos\phi_1\right) \quad \ldots (13)^*$$

Arch hinged at ends:

$$P_0 = \text{thrust at crown} = p\,r - \frac{2\,\dfrac{k^2}{r^2}\,(p\,r)\sin\phi_1}{\phi_1\,(2 + \cos 2\,\phi_1) - \dfrac{3}{2}\sin 2\,\phi_1 + \dfrac{k^2}{r^2}\left(\phi_1 + \dfrac{1}{2}\sin 2\,\phi_1\right)} \quad \ldots\ldots (18)^\ddagger$$

$$M_0 = \text{moment at crown} = -\,(p\,r - P_0)\,r\,(1 - \cos\phi_1) \quad \ldots\ldots (3)\S$$

$$P_1 = \text{thrust at abutment} = p\,r - (p\,r - P_0)\cos\phi_1 \quad \ldots\ldots\ldots (4)$$

$$M_1 = \text{moment at abutment} = 0.$$

The stresses s, in pounds per square inch, at the extrados and intrados at the crown and the abutments were computed by substituting the values of thrust and moment in the well-known formula:

$$s = \left(\frac{P}{t} \pm \frac{6\,M}{t^2}\right) \div 144$$

It should be noted regarding these formulas for thrust and moment that (*a*) they did not include the influence of shear; (*b*) they were referred to the center line of the arch ring and not to its neutral axis; and (*c*) the water pressure, p, was at the center line of the arch ring and not at the extrados.

A separate formula for shear, S (Equation (14))‖ was included in the body of the paper, several valuable observations on that subject followed in the discussion, and in his closure,¶ Professor Cain developed a more detailed formula for the value of $(p\,r - P_0)$, including shear.

The mathematical treatment of the problem was so clear and complete that the formulas were accorded wide acceptance and were extensively used (either

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 237.
† *Loc. cit.*, p. 236.
‡ *Loc. cit.*, p. 242.
§ *Loc. cit.*, p. 236.
‖ *Loc. cit.*, p. 238.
¶ *Loc. cit.*, p. 269.

with or without supplementary studies for shear). Subsequently, Professor Cain* developed further formulas for stresses in circular arches under normal loads. The more rigorous method here adopted differed from the original in the following important particulars: (*a*) The formulas for moment and thrust included the influence of shear; (*b*) both moment and thrust were referred to the neutral axis of the arch ring; and (*c*) water pressures were referred to the extrados.

The revised notation† adopted by Professor Cain was, as follows:

"The arch to be considered is a horizontal circular arch of constant radial thickness, t, and a vertical height of 1 ft., fixed at the ends, and subjected on the extrados to a water pressure of p_e lb. per sq. ft. Generally, this is not the full water pressure at the level of the medial plane of the arch, but the part carried by the arch, the remainder being carried by the supposed cantilever.

"In Fig. 2, representing the half arch,

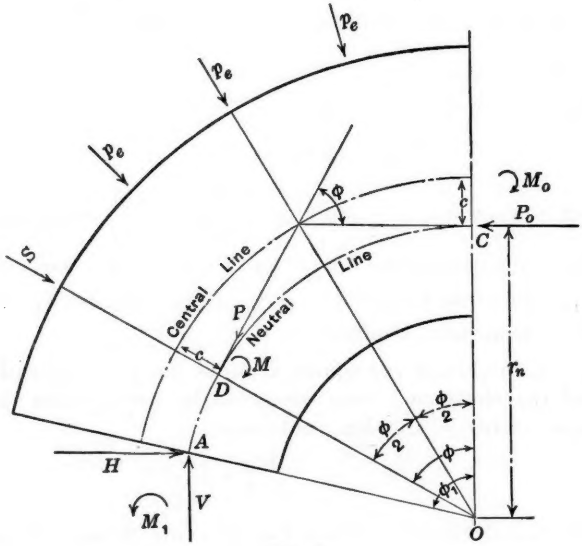


FIG. 2.

t = uniform radial thickness of arch, in feet;
r = radius of the center line, in feet;
r_n = radius of neutral line, in feet;
r_e = radius of extrados, in feet;
r_i = radius of intrados, in feet;
p_e = normal radial pressure, in pounds per square foot, on the extrados;
ϕ = angle with radius of crown for any point, D;
ϕ_1 = one-half the central angle = $A\ O\ C$.

"The arc, $A\ D\ C$, is the neutral line, so that if s = length of the arc, the distance $C\ D = r_n\ \phi$, $d\ s = r_n\ d\ \phi$.

* "Stresses in Thick Arches of Dams," by B. F. Jakobsen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 475, containing a discussion and solution by the method of least work, by William Cain, M. Am. Soc. C. E., p. 522.

† *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), pp. 528–529.

P_0 = thrust at the crown, acting at C, on the neutral line;
M_0 = moment at crown, taken positive when clockwise;
M, P, and S = respectively, the moment, normal component, and shear on the section through D $(r_n\ \phi)$;
$c = r - r_n$;***"

The revised formulas for thrust and moment developed with this notation are:

$$P_0 = \text{thrust at crown} = p_e\, r_e - X \qquad \text{(103)*}$$

$$M_0 = \text{moment at crown} = -\,X\, r_n \left(\frac{1 - \sin \phi_1}{\phi_1}\right) \qquad \text{(105)*}$$

$$P_1 = \text{thrust at abutment} = p_e\, r_e - X \cos \phi_1 \qquad \text{(99)†}$$

$$M_1 = \text{moment at abutment} = +\,X\, r_n \left(\frac{\sin \phi_1}{\phi_1} - \cos \phi_1\right) \qquad \text{(106)*}$$

In all cases,

$$X = \frac{p_e\, r_e}{D_n}\, 2 \sin \phi_1 \frac{k^2}{{r_n}^2}$$

Stresses, in pounds per square inch, for the extrados and intrados of the crown and the abutment are then computed by the following formulas:

At the crown:

$$s_e = \frac{M_0}{I} \frac{\frac{t}{2} + c}{r_e}\, r_n - \frac{P_0}{r_e \log_e \left(\frac{r_e}{r_i}\right)}$$

$$s_i = -\,\frac{M_0}{I} \frac{\frac{t}{2} - c}{r_i}\, r_n - \frac{P_0}{r_i \log_e \left(\frac{r_e}{r_i}\right)}$$

At the abutment:

$$s_e = \frac{M_1}{I} \frac{\frac{t}{2} + c}{r_e}\, r_n - \frac{P_1}{r_e \log_e \left(\frac{r_e}{r_i}\right)}$$

$$s_i = -\,\frac{M_1}{I} \frac{\frac{t}{2} - c}{r_i}\, r_n - \frac{P_1}{r_i \log_e \left(\frac{r_e}{r_i}\right)}$$

$$\text{(107)‡}$$

The complete analysis of stresses at the crown and abutments of a dam, by any of these formulas, involves a large expenditure of time.

Even neglecting shear, the conditions are so complex that analysis by the rigorous mathematical methods results in complicated equations, the solution of which is a slow and laborious process. This fact was clearly recognized by

* *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 531.

† *Loc. cit.*, p. 529.

‡ *Loc. cit.*, p. 533.

Professor Cain, who strove to lighten the labor by including in his first paper, previously mentioned, a tabulation giving the values of the complicated variable expression, $\frac{2\,\phi_1}{D_0}$. Originally the tabulation* included values for this term corresponding to five different central angles and seven different values of $\frac{t}{r}$; in the discussion, B. F. Jakobsen, M. Am. Soc. C. E., plotted the results of the tabulation, somewhat amplified, in a chart† showing, on a semi-logarithmatic scale, curves for central angles varying from 40 to 180° by steps of 10°, and for values of $\frac{t}{r}$ ranging from 0.02 to 0.30; in his closure, Professor Cain further amplified the data by presenting them in both tabular and diagrammatic‡ form. This later table covers twenty-seven values of central angle (40 to 55° by steps of 2.5°; 55 to 130° by steps of 5°; and 130 to 180° by steps of 10°); in the diagram mentioned (Fig. 10),‡ in order to avoid complication, it was necessary to omit some of the curves, but, notwithstanding these omissions, the curves included facilitate the determination of any value of $\frac{2\,\phi_1}{D_0}$ for central angles ranging from 40 to 180°, and of $\frac{t}{r}$ from 0.02 to 0.30.

Even with these aids the computation of stresses at four points (the extrados and intrados at the crown and the abutment) in a single arch ring requires:

1.—Abstracting from tables five trigonometric functions or constants; and

2.—Performing thirty or more arithmetical computations (multiplications, divisions, additions, and subtractions), involving both positive and negative quantities and numbers varying from millions down to ten thousands.

Not only are these various computations laborious, but they offer many chances for arithmetical errors, and no opportunity for checks save by duplicate computation. The revised formulas, including the effect of shear, published in the discussion on Mr. Jakobsen's paper on "Stresses in Thick Arches of Dams",§ are even more rigorous in their treatment and, hence, more complicated.

The curves presented in this paper resulted from an attempt to simplify the use of these valuable contributions to a complex engineering problem.

Due to the number of variables it would appear, at first glance, impossible to devise a simple graphic solution. For example, practically all dam sites decrease in width from top to bottom. The dam itself increases in thickness

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), Table 2, p. 246.

† *Loc. cit.*, Fig. 8, p. 257.

‡ *Loc. cit.*, Table 6, p. 266, and Fig. 10, p. 267.

§ *Loc. cit.*, Vol. 90 (June, 1927), pp. 522–601, inclusive.

from top to bottom in order to withstand the head, which increases in like manner. The central angle of the arch may vary in practice from, say, 40 to 130°, or more. The radius varies with the width of the site at any given level and with the central angle. It is obvious, therefore, that with the different conditions encountered at various elevations in any structure there is no direct relation between the stresses at any two given levels.

If, however, an imaginary arch dam is assumed, of fixed central angle and radius and uniform thickness throughout its entire height, the stresses in the arch rings at any two levels will be in proportion to the heads, and the computed stresses for a convenient head—say, 10 ft.—when multiplied by 10, will give the stresses for the arch ring under 10 $\times$ 10, or 100-ft. head; this relation permits the use of a single set of computations as a measure for the stresses at all heads.

The value of r used in such computations is immaterial, since when the head and the central angle are equal, the stresses in any two arch rings remain equal and constant so long as the ratio, $\frac{t}{r}$, remains constant—even if the length of the radius of each ring is freely changed. It is this constant relation of stresses to values of $\frac{t}{r}$ that makes it possible to show the formulas graphically.

In preparing the curves the stresses at extrados and intrados of crown and abutment of an "imaginary" arch were computed for a head of 10 ft., for central angles ranging from 40 to 180°, and for varying ranges in the values of $\frac{t}{r}$ depending on the formulas.

The three groups of curves are based on the three sets of formulas developed by Professor Cain:

Group I.—Arches with "fixed ends" (neglecting shear). Formulas developed on pages 235 to 238, *Transactions,* Am. Soc. C. E., Vol. LXXXV (1922); this set of curves (Figs. 3, 4, 5, and 6) covers central angles ($2\,\phi_1$) ranging from 40 to 180°, and values of $\frac{t}{r}$ ranging from 0.02 to 0.30. The curves for values of $\frac{t}{r} = 0.10$ to 0.30, are included, however, only for comparison with curves of Group III, which are more accurate since they include the influence of shear.

Group II.—Arches with "hinged ends" (neglecting shear). Formulas developed on pages 241 to 243, *Transactions,* Am. Soc. C. E., Vol. LXXXV (1922); this set (Figs. 7, 8, and 9) covers the same range and values of $\frac{t}{r}$. The curves for the higher values of $\frac{t}{r}$ are included, however, only for purposes of comparison since the condition, "hinged ends", is probably realized only in very thin arches; that is, arches having very small values of $\frac{t}{r}$.

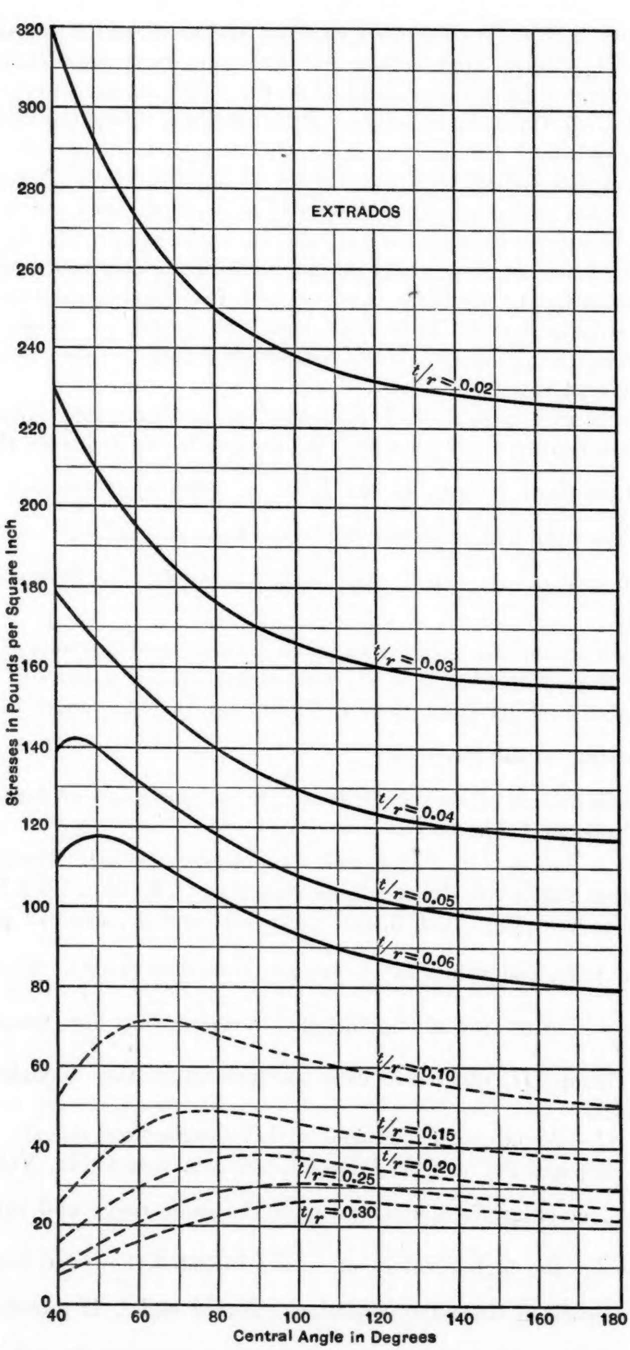


FIG. 3.—STRESSES AT CROWN—EXTRADOS. FOR 10-FOOT HEAD.

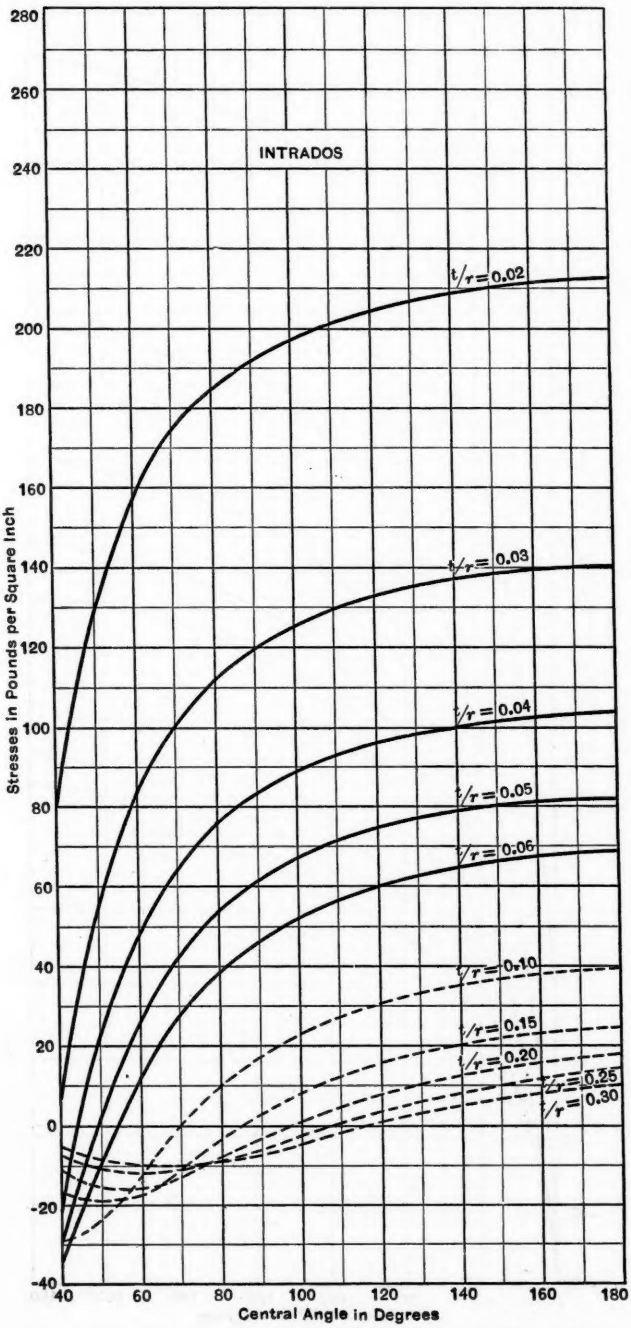


FIG. 4.—STRESSES AT CROWN—INTRADOS. FOR 10-FOOT HEAD.

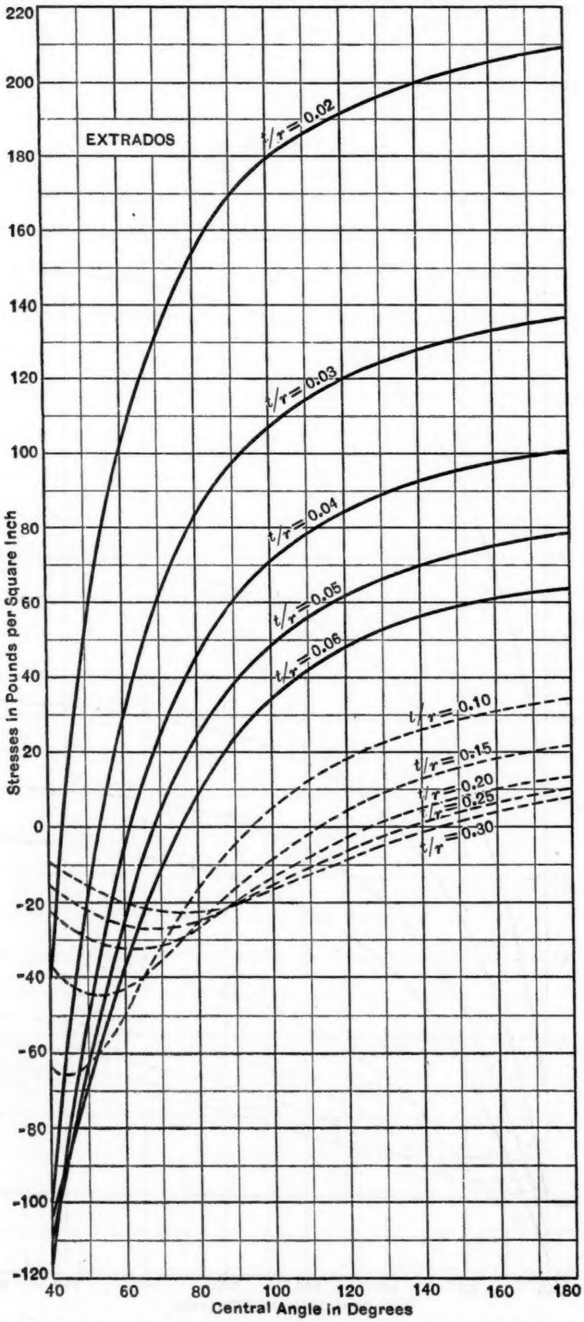


FIG. 5.—STRESSES AT ABUTMENTS—EXTRADOS. FOR 10-FOOT HEAD.

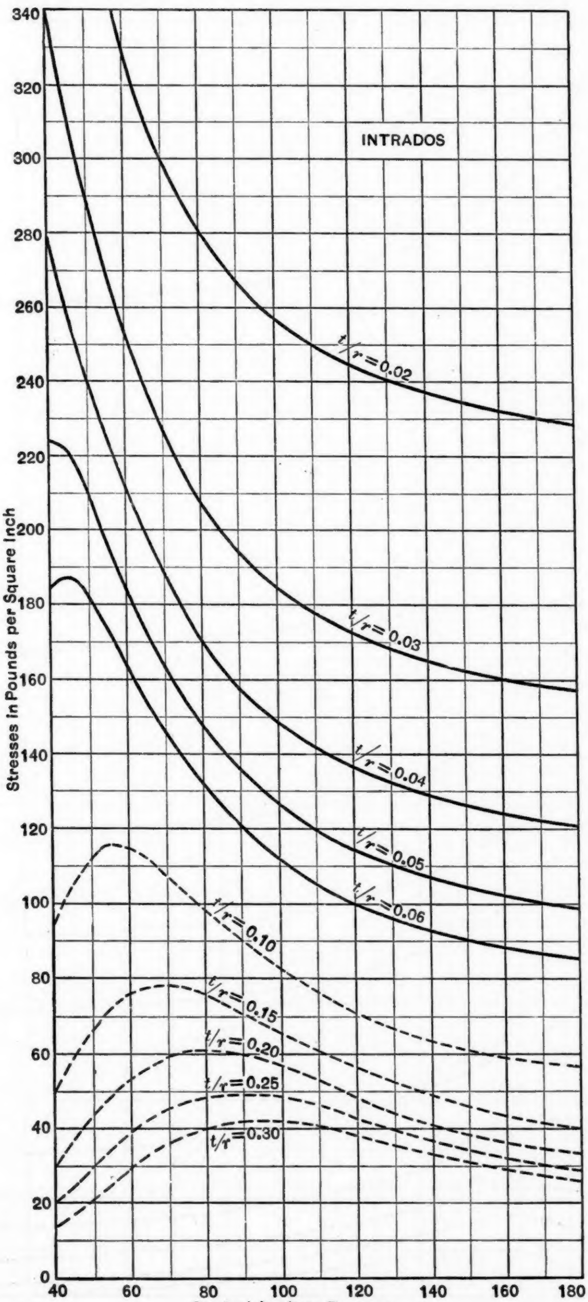


FIG. 6.—STRESSES AT ABUTMENTS—INTRADOS, FOR 10-FOOT HEAD.

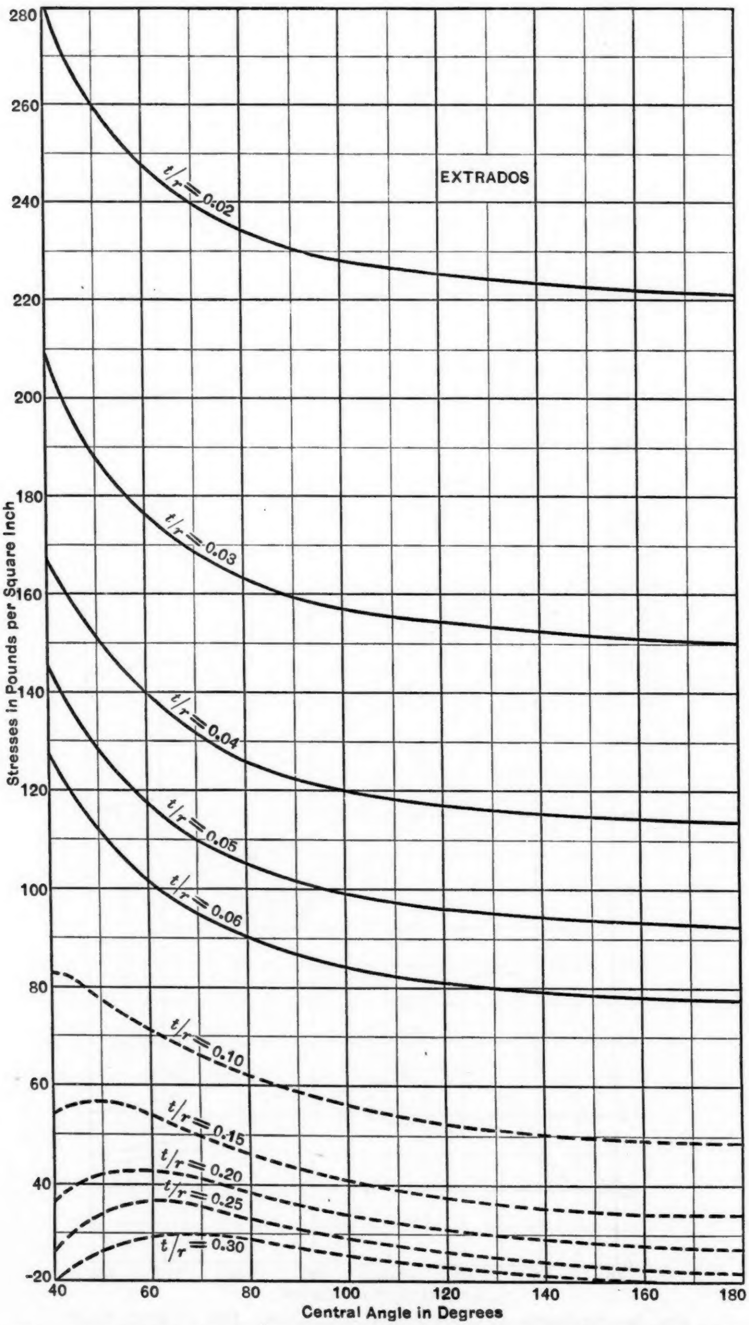


FIG. 7.—STRESSES AT CROWN, HINGED ENDS—EXTRADOS. FOR 10-FOOT HEAD.

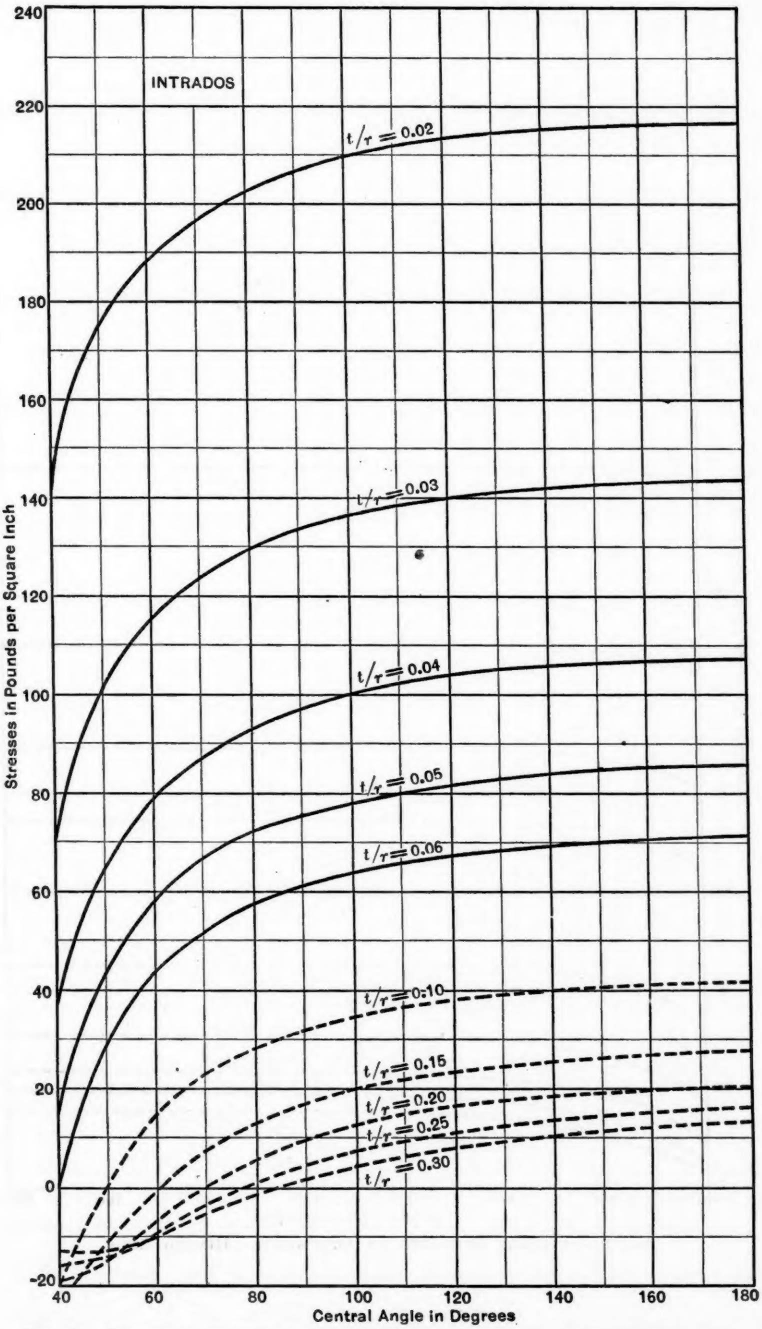


FIG. 8.—STRESSES AT CROWN, HINGED ENDS—INTRADOS. FOR 10-FOOT HEAD.

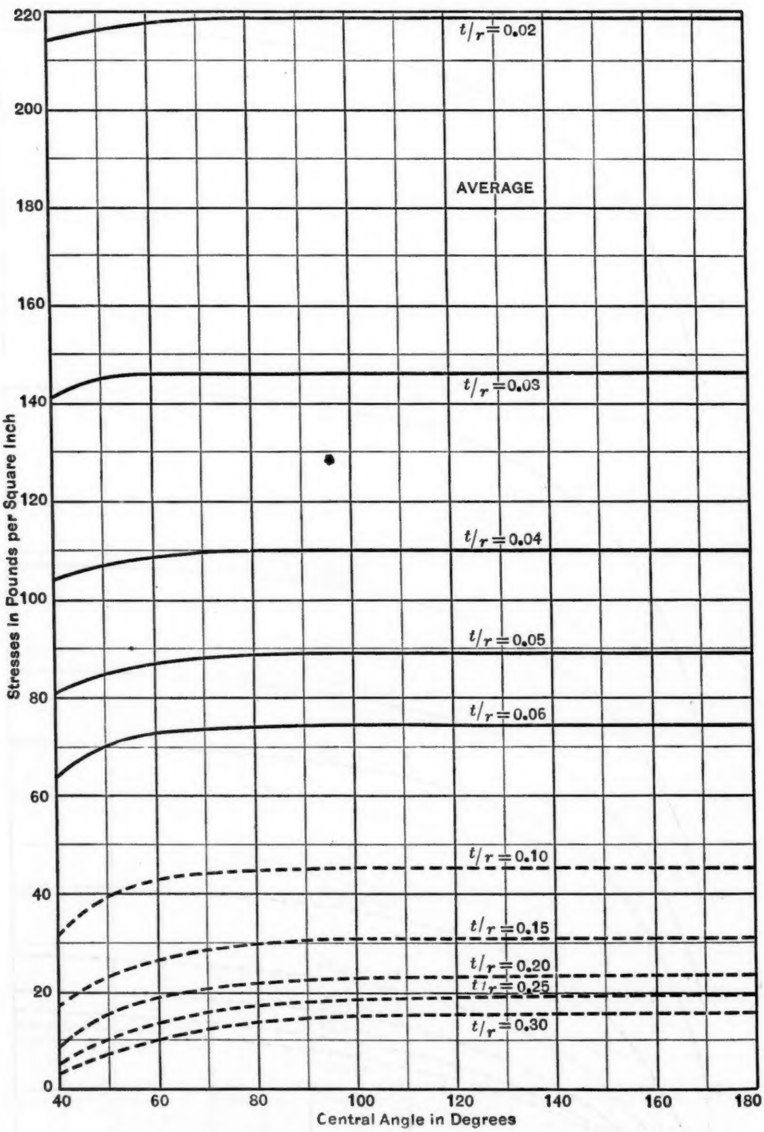


FIG. 9.—AVERAGE STRESSES AT ABUTMENTS—HINGED ENDS.

Group III.—Arches with "fixed ends" (including influence of shear). Formulas developed by the principle of "least work" on pages 522 to 534, *Transactions*, Am. Soc. C. E. (Vol. 90), June, 1927; this set of curves (Figs. 10(a) and 10(b); 11(a) and 11(b); 12(a) and 12(b); and 13(a) and 13 (b)) covers central angles ranging from 40 to 180°, and values of $\frac{t}{r}$ from 0.10 to 1.00. The curves of this group should be used in preference to those of Group I which duplicated the curves for $\frac{t}{r} = 0.10$, 0.15, 0.20, 0.25, and 0.30; these curves were included in Group I only for purposes of comparison.

Compression in all cases is shown positive (+); and tension, negative (—).

The curves of Group III, including shear, should be used throughout their range in preference to the curves of Group I, which do not include shear; the curves of Group I can be used without great error for the smaller values of $\frac{t}{r}$ (0.02 to 0.06), not covered by Group III.

Methods of Using Curves

The curves may be used with equal facility for either (a) design, or (b) analysis.

Design

Problem.—At a given level in a proposed dam it is desired to use an arch ring with a central angle of 120° and a radius of 200 ft., the maximum head being 50 ft., and the ends being assumed as "fixed". What arch thickness will be required if the maximum allowable stresses are: Compression, 500 lb. per sq. in.; tension, 0?

Solution.—Since the curves are for 10-ft. head, or one-fifth of the actual head of 50 ft., the allowable stresses to be read from the curves must be reduced in the same proportion, giving compression, 100 lb. per sq. in., and tension, 0.

The conditions and constants assumed fall in the range of curves of Group I. Since the greatest compression is always found at the intrados of the abutment, and the greatest tension at the extrados of the abutment, it is best in designing to enter Fig. 6 of this group (or Fig. 13 of Group III) first, and having determined the value of $\frac{t}{r}$, to check it against Fig. 5 (or Fig. 12, in Group III) to see whether there is a resulting tension. If tension is found, another central angle will have to be tried; if no tension is shown, the stresses at all four points are found from the proper diagrams.

In the present case it is found that the curve, $\frac{t}{r} = 0.06$, passes through the intersection of the vertical line for 120° and the horizontal line for 100 lb. per sq. in.; checking by Fig. 5, the stress horizontally opposite

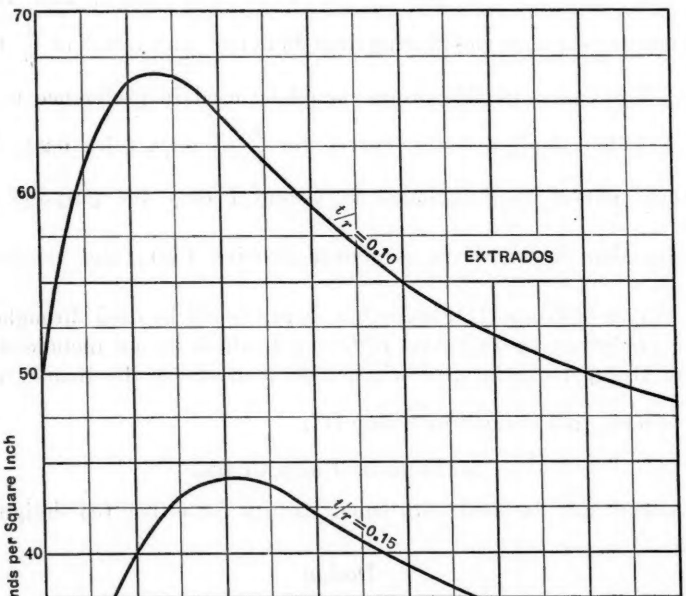


FIG. 10(a).—STRESSES AT CROWN—EXTRADOS. FIXED ENDS, INCLUDING SHEAR.

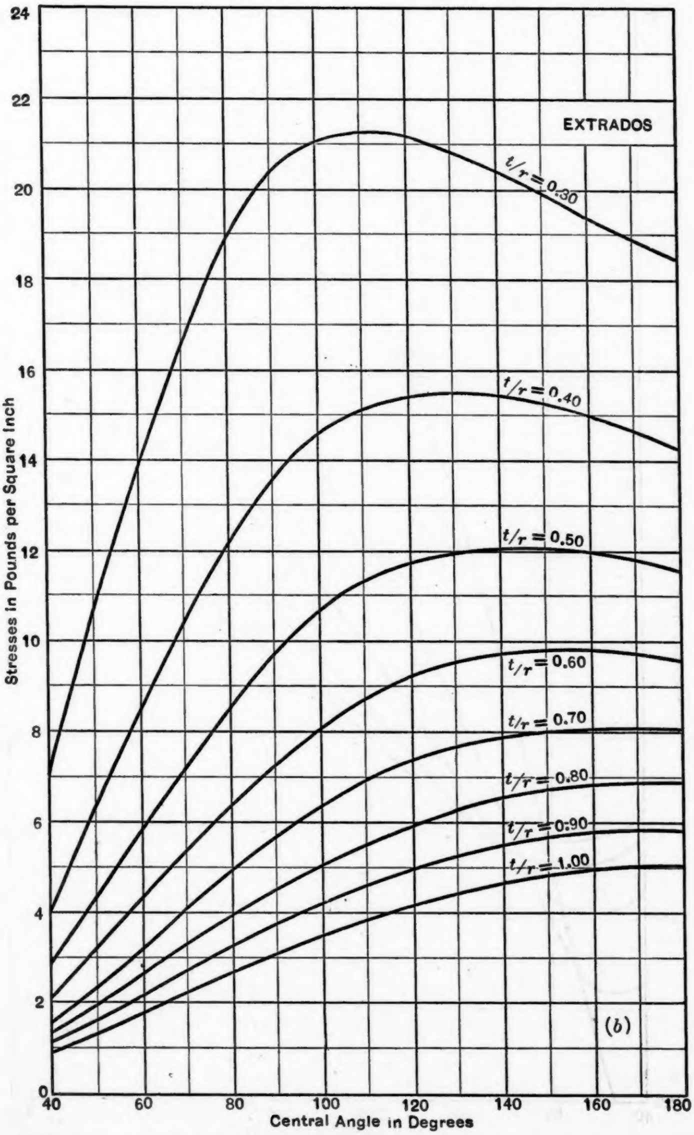


FIG. 10(*b*).—STRESSES AT CROWN—EXTRADOS. FIXED ENDS, INCLUDING SHEAR.

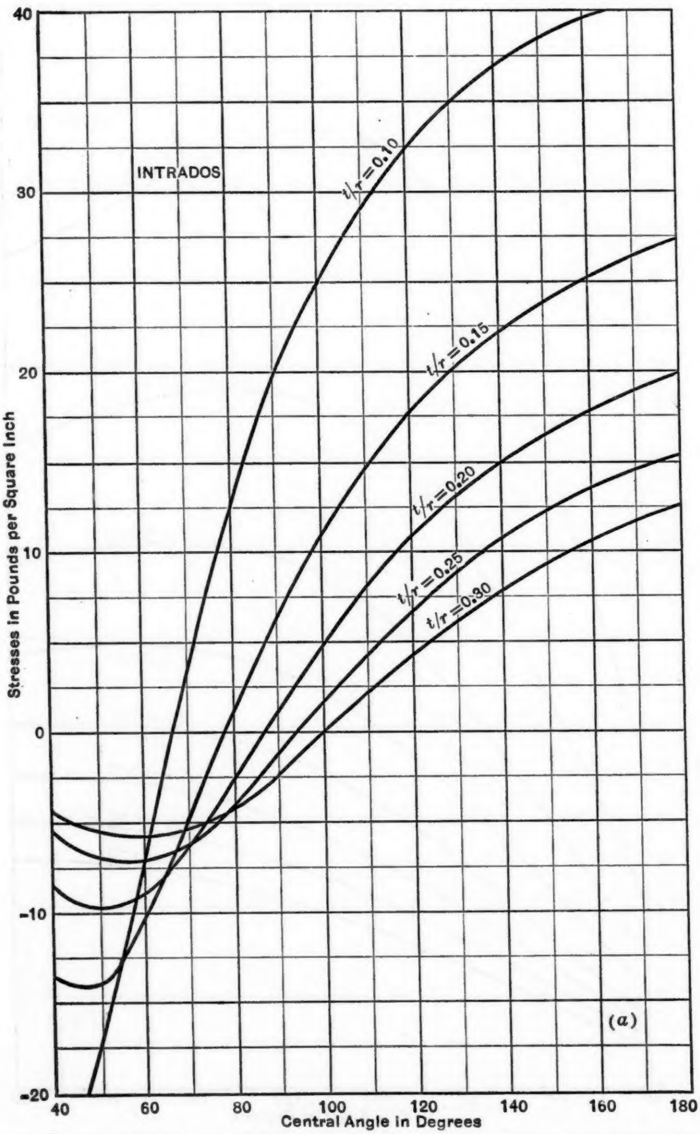


FIG. 11(*a*).—STRESSES AT CROWN—INTRADOS. FIXED ENDS, INCLUDING SHEAR.

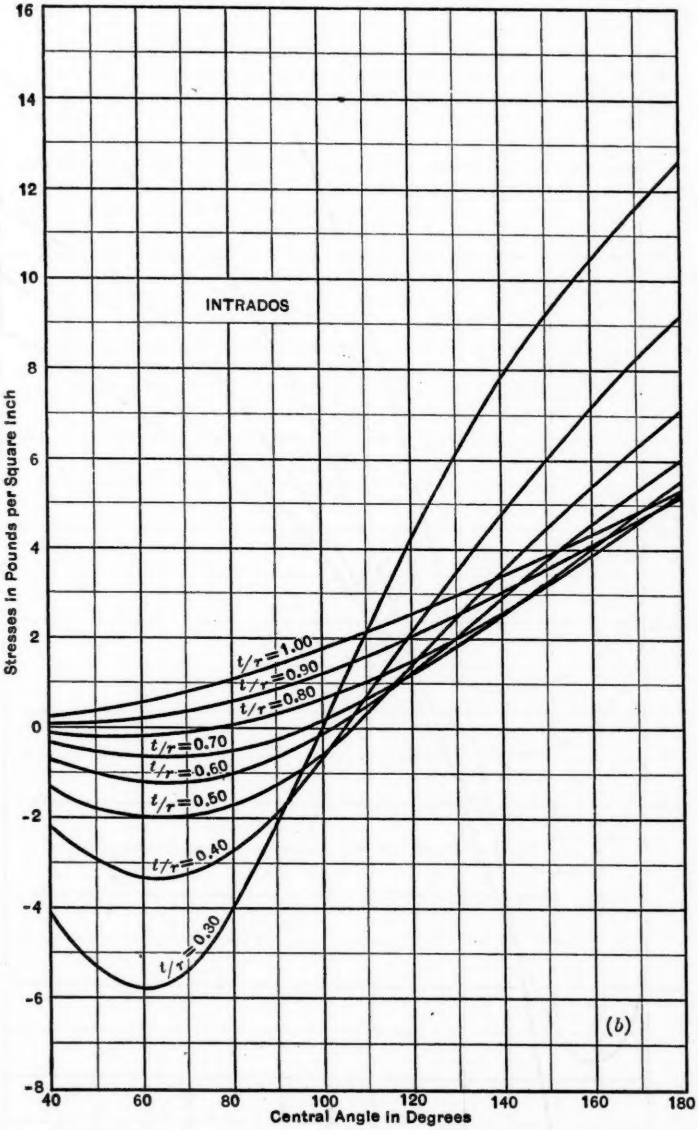


FIG. 11(b).—STRESSES AT CROWN—INTRADOS. FIXED ENDS, INCLUDING SHEAR.

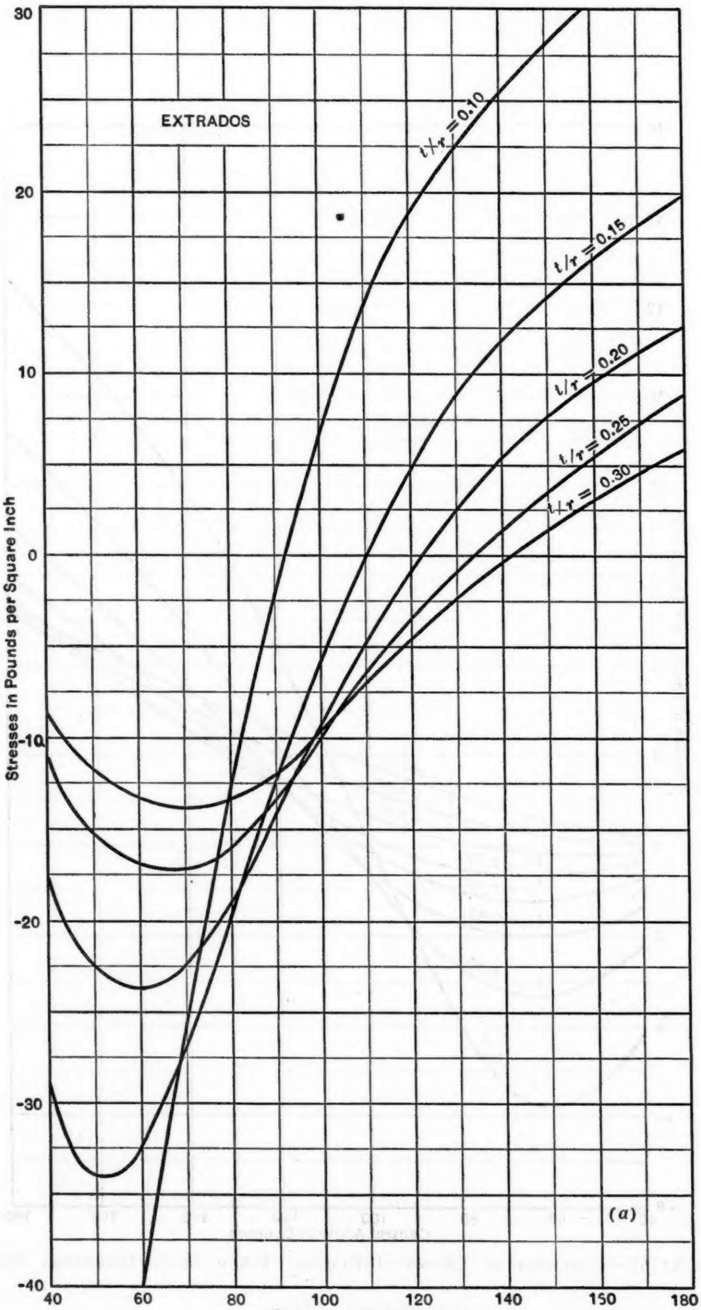


FIG. 12(a).—STRESSES AT ABUTMENTS—EXTRADOS. FIXED ENDS, INCLUDING SHEAR.

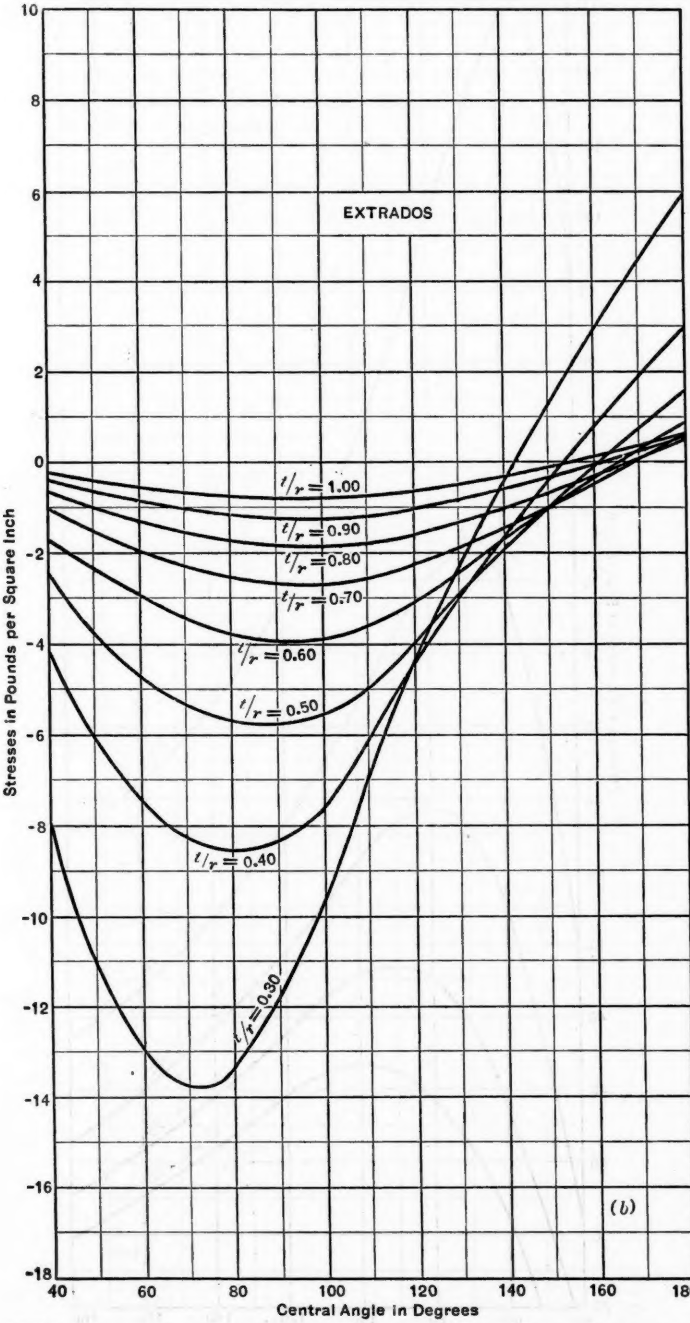


FIG. 12(b).—STRESSES AT ABUTMENTS—EXTRADOS. FIXED ENDS, INCLUDING SHEAR.

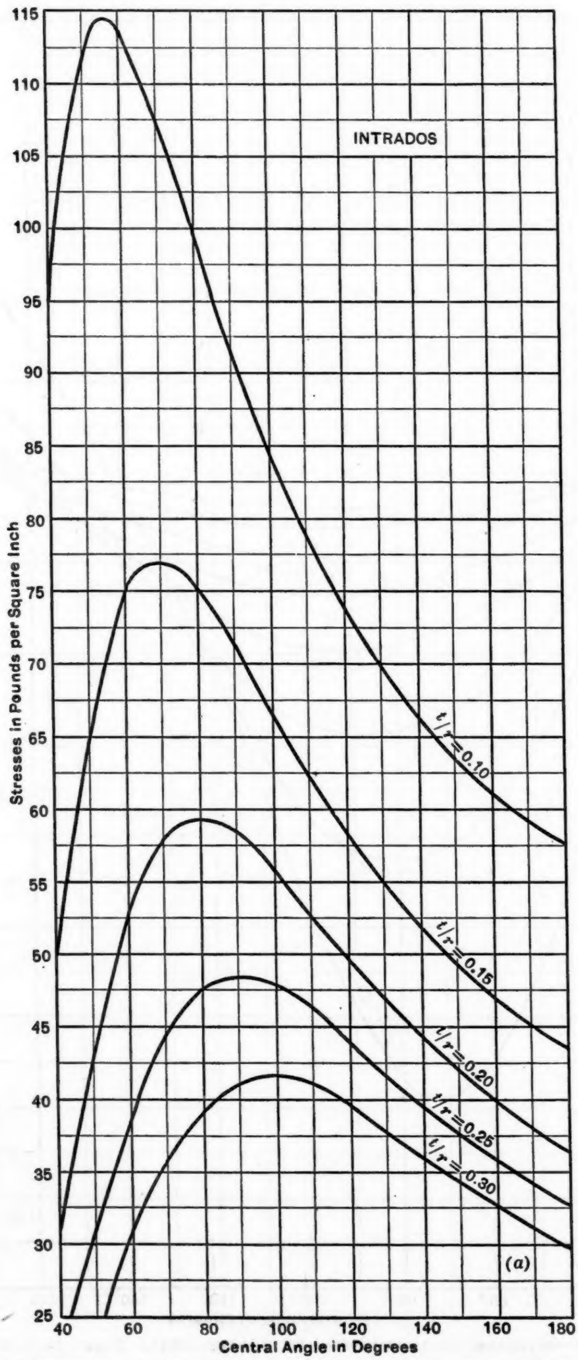


FIG. 13(a).—STRESSES AT ABUTMENTS—INTRADOS. FIXED ENDS, INCLUDING SHEAR.

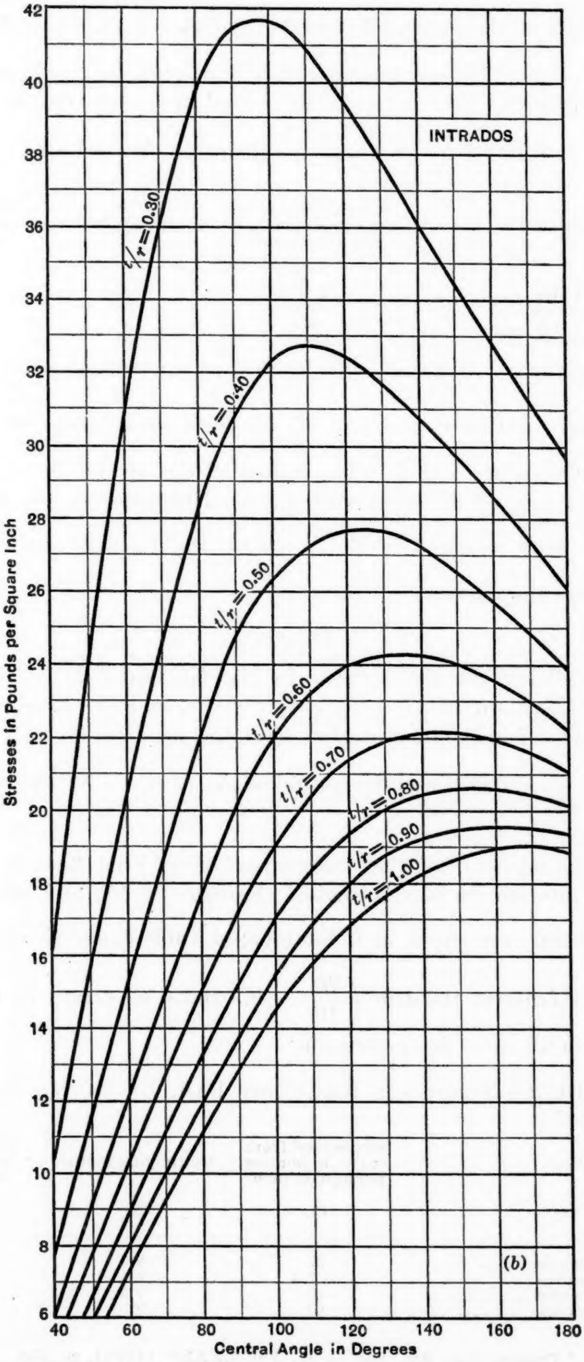


FIG. 13(b) —STRESSES AT ABUTMENTS—INTRADOS. FIXED ENDS, INCLUDING SHEAR.

the intersection of the vertical from 120° and the curve, $\frac{t}{r} = 0.06$, is + 48 lb. per sq. in., or 48 lb. compression, which satisfies the condition that there shall be no tension. The stresses in the actual arch are given in Table 1.

TABLE 1.—STRESSES IN PROPOSED ARCH DAM.

Point.		Stresses read from diagram, in pounds per square inch.	Multiplying factor.	Stresses in arch, in pounds per square inch.
Abutment:	Intrados	100	5	500
	Extrados	48	5	240
Crown:	Intrados	60	5	300
	Extrados	87.5	5	438

The radius on the center line of the arch ring being 200 ft. and $\frac{t}{r}$, 0.06, the thickness, t, is 12 ft. The radius of the up-stream face (extrados) is 206 ft. and that of the down-stream face (intrados) is 194 ft. Had the stress and $\frac{t}{r}$ been assumed in the problem, the required angle could have been read from the curves.

Analysis

Problem.—What are the stresses in the Kerckhoff Dam (already constructed) at Elevation 900, (*a*) neglecting shear, and (*b*) including shear?

Measurements* show that at that level the head is 95 ft., $t = 28.9$ ft., $r = 143.3$ ft., $\frac{t}{r} = 0.201$, and the central angle, $(2\ \phi_1) = 82° \ 40'$.

Solution.—

(*a*) Neglecting Shear.—The stresses read from Figs. 3 to 6, horizontally opposite the intersection of the vertical through 82° 40′, and an interpolated curve, $\frac{t}{r} = 0.201$, are given in Column (2) of Table 2, and the actual stresses in the given section of the dam $\left(\frac{95}{10} = 9.5 \text{ times as great}\right)$ in Column (3). Minus signs in all cases denote tension.

TABLE 2.—STRESSES IN KERCKHOFF DAM, NEGLECTING SHEAR.

Point. (1)		Stress read from curve, in pounds per square inch. (2)	Multiplying factor. (3)	Stress in dam, in pounds per square inch. (4)
Crown:	Extrados	37	9.5	352
	Intrados	− 7.5	9.5	− 71+
Abutment:	Extrados	−26	9.5	−247
	Intrados	60	9.5	570

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 258.

(*b*) Including Shear.—The stresses read from Figs. 10(*a*) to 13(*a*), horizontally opposite the intersection of the vertical through 82° 40′ and an interpolated curve through $\frac{t}{r} = 0.201$, are given in Column (2), and the resulting stresses under 95-ft. head in Column (4), of Table 3.

TABLE 3.—STRESSES IN KERCKHOFF DAM, INCLUDING SHEAR.

Point. (1)		Stress read from curve, in pounds per square inch. (2)	Multiplying factor. (3)	Stress in dam, in pounds per square inch. (4)
Crown:	Extrados	32.3	9.5	307
	Intrados	− 1.6	9.5	− 15
Abutment:	Extrados	−17.4	9.5	−165
	Intrados	59.0	9.5	560

The solutions represented by the curves of Group III were made on the basis of m (in Poisson's ratio) $= \alpha$; σ_x (vertical unit stress on the arch ring) $= 0$; and $I_n = I$.*

All the curves were computed on the basis of the arch ring at 10-ft. depth taking the full water load; in using them to compute the stresses in a ring at, say, 100-ft. depth, carrying the full water load, the unit stresses shown by the curves should be multiplied by 10; but if it is assumed that the ring at the 100-ft. depth carries only 0.8 of the water load (the remainder being carried by the vertical cantilever), the stresses read from the curves should be multiplied by 8 and not by 10.

It is possible that future experimental data secured from actual structures may lead to some modification of the Cain formula, but in the meantime it is hoped that the diagrams presented herewith may prove of practical assistance to those using Professor Cain's most valuable method of analysis.

* For comparison of unit stresses under different assumptions, by the Cain and by the Jakobsen formulas, see *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), Table 10, p. 540.

SU

com
acti
effo
not
met
was

rep
wri
dire
rep
gra
are
gra
war
the
ma
car
sec

ma
In
ser
rap
to

Oct

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

SURVEYING AND MAPPING IN THE UNITED STATES*

By C. H. Birdseye,† M. Am. Soc. C. E.

The history of surveying and mapping is the history of civilization. A complete collection of maps would be the best kind of a record of man's activities on earth, but it would be a bulky volume. Just as all early industrial efforts were crude, so were early maps—some of them mere sketches that would not be a credit to a modern school boy. As civilization progressed, however, methods and instruments were improved and the art of surveying and mapping was developed to keep pace with man's need for more accurate records.

At least as far back as 3000 B. C., man discovered that some sort of graphic representation of the earth's surface was a far more reliable record than written or oral description. The first surveys were guesses as to distance and direction. The first maps were of small areas and were pictorial in form, representing features by means of drawings. These maps had little geographic value but as the need for more accurate data developed, maps of larger areas were made and features were represented in more nearly correct geographic position. During this period the world was continually engaged in warfare and success in wars of conquest was based largely on knowledge of the fields of operation. Consequently from the very beginning maps were made mostly for military and naval activities. In fact, the history of mapping can be divided into two major periods, the first for military purposes, and the second for economic or engineering uses.

Prior to the Seventeenth Century little effort was made to show relief, and map data were confined to representation of the natural and artificial features. In 1674 a map of Paris and vicinity was made on which relief was represented by hachures. The first use of contour lines is recorded in 1728, and rapid development in the portrayal of comparative relief followed in response to a need for more accurate military data.

Note.—Written discussion on this paper will be closed in **February, 1928.**

* Presented at the meeting of the Surveying and Mapping Division, Philadelphia, Pa., October 6, 1926.

† Chf. Topographic Engr., U. S. Geological Survey, Washington, D. C.

Early in European history fragmentary surveys around fortified places were connected by other surveys, and a systematic plan was adopted to survey and map the entire country, so that, to-day, most of the European nations have maps of their territory adequate for most military and civil needs. It is surprising that the United States did not profit by the experience of Europe and start a systematic program of surveying and mapping immediately after the Revolution. To be sure, one of the first problems was to dispose of the public land, and as early as 1796 Congress authorized the appointment of a Surveyor General, under whose direction public land surveys were started in what is now the State of Ohio. Soon after this work was begun Congress directed the Surveyor General to "cause a fair plat to be made of the townships and fractional parts of townships contained in the lands, describing the subdivisions thereof and the marks of the corners." This direction has resulted in the preparation of plats of all the surveyed public lands, but these plats are not much more than graphic records of the surveyed lines. If those responsible for the early planning of public land surveys had used a little more foresight, adequate topographic maps of most of the original public land in the United States would now be completed.

The two periods in the history of American mapping can be divided roughly at the time of the Civil War. Before that the surveys were largely for military purposes, but afterward the exploratory surveys were inaugurated purely for scientific and economic study.

The need for topographic maps was recognized during the Revolutionary War, and on July 22, 1777, Congress authorized the appointment of a "geographer and surveyor of the roads, to take sketches of the country and the seat of war." On March 3, 1813, Congress authorized the appointment of eight topographic engineers and eight assistant topographic engineers under the direction of the General Staff of the Army. These officers formed the nucleus of the first Army Corps of Topographic Engineers. The Corps continued to function as an independent unit until the Civil War, but early in 1863 it was absorbed by the Corps of Engineers.

Before the Civil War many expeditions under the direction of officers of the Army explored the vast territory lying west of the Mississippi River. At first, the instruments and methods used were rather crude, but later improvements gradually resulted in better maps. However, practically none of the results secured prior to the Civil War could be incorporated in the standard map of the United States. During that period several systematic surveys were started by the Federal Government and by some of the Eastern States. The most far-reaching move was the creation of the United States Coast and Geodetic Survey in 1816. This organization made an accurate hydrographic and topographic survey of a narrow strip along the Atlantic Coast line, and in this connection established the first accurate system of triangulation in the United States. Its surveys included the topographic mapping of the principal rivers entering the Atlantic Ocean. Relief was shown by hachures. Not until 1846, when it mapped an area around the Harbor of Boston did the Coast Survey use contour lines for representation of vertical heights. This, how-

ever, was not the first contour map made in the United States, for in 1835 the Geological Survey of Maryland issued a contour map of a small area that is believed to be the first American map of this kind.

At the outbreak of the Civil War all surveys, except those needed for military purposes, ceased; but after the war the territory west of the Mississippi River again became the center of mapping activity and many exploratory surveys were carried on by the Federal Government and by universities, railroad companies, etc. The most important of the Government surveys were as follows:

1.—The U. S. Geological and Geographical Survey of the Territories, directed by Professor F. V. Hayden, from 1867 to 1879, under the Interior Department.

2.—The geographical surveys west of the 100th meridian, directed by Capt. George M. Wheeler, from 1868 to 1879, under the War Department.

3.—The U. S. Geographical and Geological Survey of the Rocky Mountain Region, directed by Major J. W. Powell, from 1869 to 1879, under the Smithsonian Institution.

4.—The U. S. Geological Exploration of the Fortieth Parallel, directed by Clarence King, from 1871 to 1879, under the War Department.

These four surveys were alike in character—all were exploratory, but they covered large areas completely and resulted in the first really comprehensive topographic maps made in America. Astronomic observations were made, base lines were measured, and an accurate system of triangulation was extended over the areas mapped. Differences of elevation were determined by vertical angles and by barometers. The representation of physiographic forms was somewhat crude as compared with present-day practice, but it was such an improvement over previous work that in the period from 1867 to 1879 more progress was made in geographic research than any other period of similar length in American history. The Hayden, Wheeler, Powell, and King Surveys ceased in 1879 when Congress created the U. S. Geological Survey.

Four Federal organizations, the Coast and Geodetic Survey, the General Land Office, the Corps of Engineers, and the Geological Survey, have continued to the present day as the principal surveying and mapping agencies.

The survey of the coasts of the United States was authorized by Congress on February 10, 1807, but no field work was undertaken until 1816. The Coast Survey was a bureau of the Treasury Department until 1903 except for a brief period under the Navy. It was then transferred to the newly-organized Department of Commerce and Labor; and, in 1913, was placed under the jurisdiction of the Department of Commerce.

For half a century its work was confined to surveys along the coasts and navigable rivers of the United States, but in 1871 Congress appropriated funds for connecting the surveys along the Atlantic Ocean with those along the Pacific Ocean by a system of triangulation. As a logical result of this action the name of the organization was changed in 1878 to the Coast and Geodetic Survey.

Practically all the coast lines of the United States, Porto Rico, Hawaii, and the Philippine Islands have been surveyed, and hydrographic charts made.

of the adjacent waters. Similar surveys and charts have been made for a considerable part of the Alaskan Coast. Because of the rapidly changing channels formed by sand-bars and shoals, resurveys are required from time to time; consequently, this work will never be completed.

The Coast and Geodetic Survey performs a work absolutely essential to the Geological Survey and other map-making organizations of the United States, for, in addition to preparing the charts of the coasts, it furnishes the main control points for all other local surveys. Its elevations and geodetic positions are surpassed in accuracy by no other organization. It computes and publishes tidal constants for the principal ports of the United States. It determines magnetic declinations, its mathematicians have devised and developed new methods for computing geodetic constants relating to the size and shape of the earth, and its investigations in isostasy are known all over the world. Its publications are numerous and invaluable to navigators and students in all lines of geodetic work.

The sub-division of the public domain into townships and sections was authorized by Congress in 1785. The first public land surveys were made in 1796 in the area north of the Ohio River and west of the Pennsylvania State line.

In 1812 the General Land Office was organized as a bureau of the Treasury Department. In 1849 it was transferred to the Department of the Interior, where it has remained until the present time. The General Land Office has jurisdiction over the survey and sale of the public lands of the United States, including Alaska.

Previous to 1910 all field surveys were made by contract surveyors, a procedure which resulted in much work of doubtful accuracy. Since 1910 the surveys have been made by regular employees of the Land Office, with resulting decrease in costs and great increase in accuracy.

The sub-division and sale of the public land require cadastral surveys only. Such topographic features as are shown on the plats are incidental to the regular cadastral work. The shores of lakes and large rivers are meandered, but, in general, the accurate location of streams, roads, trails, and other natural and artificial features are determined only at the crossings of the public land lines, and the intermediate data are sketched on the plats. Relief is shown generally by hachure lines, but in a few special surveys contour lines based on accurate elevations have been used. The horizontal control is furnished by a system of base lines, principal meridians, standard parallels, and guide meridians, but only recently has an effort been made to tie these control lines to the standard triangulation scheme of the United States.

Public land surveys have been carried on in all States except the thirteen original States and Tennessee, Kentucky, and Texas. Seventeen of the older States have been completely surveyed, but as some of the early surveys were inaccurate or insufficiently marked, parts of these States are being resurveyed at the rate of about 1 000 000 acres per year. Original surveys are being extended annually over about 2 000 000 acres.

The Corps of Engineers of the Army was amalgamated with the Corps of Topographic Engineers of the Army in 1863, and in the early days engineer officers were the surveying pioneers in the West. Since that date the Corps has executed or supervised practically all the mapping operations of the Army. Prior to the entry of the United States into the World War the mapping operations of the Corps of Engineers were usually reconnaissance in nature, except on river and harbor and fortification projects, and in the Philippines, Hawaii, and Panama. However, experience in the World War demonstrated the importance of accurate topographic maps for all kinds of military operations (a fact always known to military engineers) and a new impetus was given to military mapping. The Corps of Engineers has trained a Topographic Battalion, and its men are engaged on accurate topographic mapping within, as well as without, the United States. Modern methods of topographic mapping form an important part of the training of all engineer troops. Especial stress is laid on full use of aeroplane photographs. The best camera for mapping purposes was developed by an engineer officer, who received his mapping training in the Geological Survey.

Close co-operation exists between the Corps of Engineers and the Geological Survey so that the mapping efforts of the Army are confined to surveys of strategic importance, to resurveys which have been inadequately mapped, and to surveys which the Geological Survey for any reason has been unable to map. Each year, a substantial part of the Army appropriation for military surveys and maps is made available to the Geological Survey for special work urgently needed by the Army.

The organic act of the Geological Survey provided that the Director "shall have the direction of the Geological Survey and the classification of the public lands and examination of the geologic structure, mineral resources, and products of the National Domain." Clarence King was the first Director. From his experience on the King Survey he realized that no adequate classification of lands and no geologic examination could be made without topographic base maps; hence in the earlier years topographic maps were made a part of the general work of the Geological Survey, and allotments of funds for this purpose were made from the general appropriation. The first specific appropriation for topographic surveys was made in the Sundry Civil Act of October 2, 1888, and amounted to $199 000. Appropriations for topographic mapping have been continued each year. The amounts have fluctuated somewhat, but reached $300 000 in 1903. During the next twenty years the appropriations remained between $300 000 and $350 000. In 1924 the appropriation was raised to $500 000 and has continued to this year (1926) at approximately that figure. Including funds allotted in the early years from the general appropriation of the Survey, approximately $13 200 000 has been expended for topographic mapping. The Geological Survey has also received special appropriations (amounts approximate) for surveying National forests, $2 200 000; for military surveys and maps, $1 400 000; and allotments by co-operating States, $5 100 000; so that the total expenditures for topographic mapping, including those for the present fiscal year (1926), amount to approximately $21 900 000.

The current Federal appropriation for topographic mapping by the Geological Survey is \$451 700; the allotment for military maps is \$12 500; and State allotments amount to \$380 000, so that the total amount available for topographic mapping during the current fiscal year is about \$844 200.

The area of the continental United States, exclusive of Alaska, is more than 3 000 000 sq. miles. In the 47 years of the Survey's existence about 43% of this area has been surveyed and the results are published in nearly 3 400 different topographic maps. Some of the early maps were made by methods so crude that the area must be resurveyed. Consequently, only about 30% of the area of the country is covered by maps that are adequate for present-day uses. Some of the States are already completely surveyed. Notable among these are Ohio, West Virginia, and Maryland, which are covered by modern maps resulting from an early adopted and well co-ordinated plan of co-operation. Other States, such as Massachusetts, Rhode Island, Connecticut, New York, New Jersey, and Delaware, are also completely surveyed, but some of the maps are old and out of date and need revision or resurvey. New York has just started a program of revision, and other States have indicated a desire to follow.

During the current year (1926), co-operation is being maintained in 24 States and Hawaii. Exclusive of the States that are completely surveyed, co-operative mapping is being carried on in all but 17, and work of purely Federal interest has been undertaken in 3 of these, so that surveys are in progress in 27 States and 1 Territory. At the past rate of progress it would take nearly 100 years to complete the mapping of the United States. That is too long a period, and American engineers have united in demanding more rapid progress, so that this generation may reap a larger benefit. After carefully considering the problem the Geological Survey proposed a 20-year program. This was approved by Congress in what is now known as the Temple Act. The Act authorizes the completion of the mapping in 20 years and contemplates appropriations gradually increasing for a number of years and then decreasing toward the end, both for geodetic and topographic work. As the Act only authorizes the first year's appropriation and does not actually appropriate any money, further legislative action is required.

The Geological Survey long ago adopted a policy of State co-operation by which those States that desired could pay one-half the cost of the work in order to expedite the mapping of the areas within their borders. The Temple Act recognizes this co-operation and contemplates the continuation of the policy. In areas in which the Federal interest is supreme, such as the National forests, National parks, and Indian reservations, the Federal Government should pay the entire expense, but in other areas the States will be expected to bear one-half the cost of the surveys.

The estimated cost of completing the mapping of the United States is \$49 200 000. Of this amount, \$4 200 000 will be spent for precise surveys—triangulation, traverse, and spirit leveling—to obtain horizontal and vertical control. This work is done by the Coast and Geodetic Survey and must be executed a year or more in advance of the mapping operations. It is estimated

that the several States, or their sub-divisions, will contribute about $12 000 000, so that the Federal cost of the mapping will be about $33 000 000. Spread over a period of 20 years this is not a large investment—not much more than the cost of a single battleship—and the investment will be more than returned to the taxpayer by direct savings made in the construction of public works, etc.

The President, by Executive order of December 30, 1919, created a Federal Board of Surveys and Maps, composed of representatives from each of the Government organizations engaged in surveying and mapping. This Board has made great progress in the co-ordination of mapping operations and in standardization of results. Little serious duplication of effort was found to exist. Most of the lack of co-ordination has been corrected—not by orders or regulation, for the Board has only advisory powers—but by bringing together into close contact the men responsible for the immediate supervision of the mapping efforts of the Federal Government. These men are brought in closer touch with the map-using public through an advisory council. This close personal contact is the secret of real co-ordination.

The Advisory Council is made up of representatives of non-Federal agencies interested in mapping. The mapping needs of the United States have been presented and plans have been made to meet them. The existence of much valuable survey data from non-Federal sources has been discovered, and these data have been made available to the Federal mapping agencies, so that now a Government project is rarely undertaken without examining all available data and using all that have merit.

The Executive order creating the Board of Surveys and Maps also directed the establishment of a central Map Information Office to classify, index, and distribute to the map-using public information regarding survey and map data. This office has proved to be of great assistance to the Federal organizations interested in mapping as well as to individuals seeking map data. The Office is not a distributing agency, but it has recorded most of the Federal map data and much non-Federal map material. Use of this Office is recommended to all who do not know whether the data they desire are in existence or where to find them if they do exist.

Among some of the noteworthy results of the work of the Board may be mentioned the following:

Standard specifications for control surveys.
Standard map symbols.
Adoption of standard projections for different types of maps.
Selection of a standard series of base maps.
Standard instructions for executing control surveys and topographic surveys.
Research in problems of aerial photography.
Establishment of an efficient Map Information Office.

No discussion of mapping is complete without reference to aerial photography. Much misleading information has been given to the public. Claims have been made in the public press that the entire area of the United States will be mapped immediately by use of aerial photographs and at a cost much lower than by ground methods. The Federal mapping agencies wish that these

claims were correct, but the truth is that aerial photography will never replace ground surveys entirely, although it will aid them, expedite them, and lessen the final cost.

Under certain conditions the use of aerial photography will eliminate most of the ground survey work. It is especially useful in flat areas that are open or not too densely timbered, such as the marsh lands in the deltas of some of the Southern rivers, where there are no roads, trails, or houses, and where a single determination of elevation may be sufficient for contouring a large region. In mapping large cities the use of aerial photography is of tremendous value. In the revision of old surveys, made necessary by changes in the works of man, aerial photographs may make it possible to publish an up-to-date map without the necessity of any resurvey by ground methods.

However, the whole of the United States will not be photographed for many years—probably never, at least for use in mapping. In certain areas the photographs would be of so little help to the map maker that the expense of making them would not be warranted for the purpose of map making alone. Any densely timbered area presents an example of conditions that are most unfavorable to photographic surveying, either from the air or from ground stations. The meanders of the streams would of course be indicated on the pictures, but the topographer must traverse these streams to determine their fall, measuring accurately differences of elevation, particularly where possibilities of power development exist. The topographer must locate every trail and house and must tie his surveys to the section and township corners of the public-land survey. The pictures can not show these features in such an area, and new processes must be perfected before aerial surveys will be of much help to topographic mapping in dense forests.

In areas of considerable relief the use of aerial photographs is of doubtful value for small-scale topographic mapping. For example, in the high Rocky Mountains the scale usually employed by the Geological Survey is 2 miles to the inch, and surveys by ground methods cost about $20 per square mile. An aerial photograph is only a perspective view from the lens of the camera, and every plane of elevation in a single picture will of course have a different scale. These differences in scale can be co-ordinated, but the process is laborious and expensive, and with present developments the cost of taking the pictures and rectifying the photographic data may exceed the cost of a ground survey. Moreover, the use of photographs of such a region requires an abundance of control points accurately located by triangulation, an operation performed by the topographer in the course of his ground survey. The topographer has to climb all over the region in any event, not only to make the control surveys, but to make the connections with the public land lines.

Up to the present time no practicable method has been devised for determining elevations and portraying contour lines on small-scale maps of large areas by the use of aerial photographs. Some who are familiar with developments in mapping from photographs may challenge this statement, but the writer knows of only one firm in the United States that has successfully solved

the problem of the accurate portrayal of contour lines from aerial photographs, and this only in surveys on a very large scale.

Aerial photography is welcomed as a supplement to ground surveys, but it does not promise to replace them to any large extent, particularly in the Western Mountain States. In 1925 the Navy Department and the Geological Survey spent considerable money in making aerial photographs of an area adjacent to the naval oil-shale reserve near Grand Valley, Colorado. The field surveys were completed before the pictures were available, and about the only use it was possible to make of them was in inspecting and checking the field mapping. In Texas, on the other hand, hardly a square mile has been mapped during the last two years without the aid of aerial photographs, thus saving about 30% of the ordinary cost.

Aerial photography has passed the experimental stage, and future improvements in apparatus and economies of procedure promise a much larger use than is now possible. It will pay to use this new method when and where the map data can be gathered by it cheaper than by surveys on the ground. It will not pay when and where the expense of taking the photographs and of reducing the data is greater than the cost of collecting the information from ground surveys. The Geological Survey will use aerial photography where it pays and will not use it when the evidence indicates that it will not save time and money.

It has been the experience of the Geological Survey that as fast as improvements are devised in methods and instruments which might be expected to cheapen the product, a demand has arisen for more detailed maps, and the increased cost of meeting this demand has offset the saving made by the new methods. Therefore, the topographic surveys now in progress in the United States may be expected to yield better maps rather than cheaper ones, because of this new aid to the topographic engineer.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS
This Society is not responsible for any statement made or opinion expressed in its publications.

SURVEYS ON THE COAST OF NEW JERSEY*

By Victor Gelineau,† M. Am. Soc. C. E.

All surveys of a coastal region should be based on the fact that shore lines shift their positions, thereby changing land ownership. This applies with particular force to New Jersey, for it is almost surrounded by water. The shore line is a property boundary and its variations, within historic times, are most interesting to the physiographer, the surveyor, and the civil engineer. Probably these changes are not in fact more extensive than have occurred in other ocean-front States, but the relatively high degree of development of the New Jersey Coast has compelled the study of shore-line variations and of measures to preclude radical change of contour. The only means of measuring the changes in shore line is by executing repeated surveys referred to a common datum.

Shifting of Shore Lines

Whenever water waves beat on a shore they tend, in general, to level down land forms. The result of tidal and wave forces is to carry beach material in one dominant direction. Thus, in New Jersey, Barnegat Inlet is approximately the division point of the shore drift. From Barnegat Inlet northward to Sandy Hook, the beach material generally tends to drift northward, while south of Barnegat the direction of the along-shore drift is southward. This is not always true, for certain storms will cause a quick reversal of drift and there are sections where local movements are almost always opposite in direction to the general movement.

The travel of this material along the beach is the primary cause of (*a*) the alteration in direction and position of the inlet gorges; and (*b*) the tendency of certain inlets to close, particularly those the lagoon areas of which are relatively small.

Examples of Shifting Inlets

The shifting and the closing of inlets and their subsequent re-opening by natural or artificial agencies and the formation of new inlets have served to

Note.—Written discussion on this paper will be closed in February, 1928.

* Presented at meeting of the Surveying and Mapping Division, Philadelphia, Pa., October 6, 1926.

† Director and Chf. Engr., New Jersey Board of Commerce and Nav., Jersey City, N. J.

complicate the location of property lines and have in fact obliterated large land holdings A series of inlets existing on the New Jersey Coast within recent times were of virtually no value from the standpoint of navigation, but were always a menace from the standpoint of coast erosion and could be rather confidently expected to close up every year or two. The inlets leading into Deal Lake, Sunset Pond, Wesley Lake, Goose Pond, Duck Pond, Silver Lake Inlet, and Sea Girt Inlet were examples in point. To-day, these inlets are all permanently closed, except for flumes or gates that permit drainage of the pond or basin.

Shark Inlet and Manasquan Inlet, the lagoon areas of which are relatively restricted, are two interesting examples of the tendency of inlets to shift and, occasionally, to close. Their behavior prior to the improvement of Shark Inlet was almost parallel. The normal position of the inlet gorge would be approximately at right angles to the general line of the beach, making a most favorable condition for the ebb and flood flows. The gorge would remain in this position for a more or less indeterminate period, but the actively moving alongshore drift of sand would gradually shift the inlet gorge to the northward so that its channel would flow that way nearly parallel to the beach for several hundred feet before swinging eastward into the ocean. Ultimately, a position would be reached, owing to the great length of the gorge, where the velocity of the tidal current would be insufficient to scour the sand out and the inlet would close. An abnormal difference in the water levels between the lagoon and the ocean, accompanied by storm conditions, might result in the inlet's gorge re-establishing itself in the normal position; or the opening might be made by dredging a ditch or a small canal across the sand-bar. Then the cycle would repeat itself.

Shark Inlet was fixed in position after 1911 when the State enacted legislation and made the necessary appropriations. Manasquan Inlet has been closed from April, 1926, to January, 1927. At the time of the U. S. War Department Survey in 1878 it was fully 1 100 ft. north of its normal position. A number of other inlets, referred to in old works and records, have been closed for perhaps 100 years or more. Their location must now be guessed by such evidence as the meadow islands and soundings will furnish. Cranberry Inlet comes within this description. There is some evidence that it was used during the War of the Revolution. Its former existence furnishes an explanation for giving the name of Island Beach to the beach that extends northward from Barnegat Inlet. Attempts were made by Michael Ortley, in 1821, and Anthony Ivins, Jr., in 1847, to re-open an inlet in this vicinity, but without success.

Triangulation Control

The foregoing demonstrates conclusively how vital it is, in surveying on the ocean front, especially wherever a yielding, mobile shore material is impinged upon by swift currents or large waves, to fix all stations accurately from reference points of reasonable permanence. Monuments should be placed well back from the high-water line and all important points should be referenced

and located by triangulation or by an adaptation of the three-point method. Three points, however, may not be enough; partly because they may be poorly located and, therefore, fail to give a good intersection, but primarily because of the probability that some of them may be destroyed within a relatively few years. In 1912, for instance, the traverse points of a survey were referenced by the three-point method to a lighthouse and two water towers, all very substantial structures. The towers were removed when the pumping plants were enlarged and, in a re-survey of 1925, the traverse monuments had to be recovered by other methods.

The writer urges an application of the triangulation system. It is almost invariably superior to traverses for surveying in the vicinity of waterways other than the ocean front. The inter-visibility of the stations, the ease with which adjustments can be made, the facility with which stations can be recovered, and the impracticability of chaining across large bodies of water, are strong arguments for its use.

Authority for Making Surveys

There is no unit of the State Government of New Jersey that applies itself primarily to the making of surveys and maps. Each State Department the operations of which require surveys (and there are several) has its own individual surveying organization to execute the surveys and plans required by that Department. The first "system" of surveys established in New Jersey includes the necessarily scattered locations of tracts granted by the ancient Proprietors of West Jersey or East Jersey. It seems advisable to explain the source of land proprietorship or title in New Jersey.

Land Titles

New Jersey was originally a Dutch Colony, settled under the auspices of the Dutch West India Company. Many years after the Dutch settlement, the activities of the Dutch marking them as dangerous enemies to Sweden, Gustavus Adolphus of Sweden, in 1638, sent out an expedition which settled on both sides of the Delaware. Prosperous for a time, the Swedish settlements weakened and, in 1655, the Dutch conquered New Sweden. From this year until 1664, New Jersey was absolutely a Dutch Colony.

On March 12, 1664, Charles the II, of England, granted to his brother, James, Duke of York, all of what is now New Jersey, as well as Long Island and other territory to the north and east. During that year the British Naval Power compelled the surrender of the Dutch claims. In the meantime (June 23-24, 1664), the Duke of York, by indenture of lease and release, conveyed to John, Lord Berkeley, and Sir George Carteret, lands which include what is now New Jersey. These are described* as follows:

"All that tract of land adjacent to New England, and lying and being to the westward of Long Island, and Manhitas Island, and bounded on the east part by the main sea, and part by Hudson's river, and hath upon the west Delaware bay or river, and extendeth southward to the main ocean as far as Cape May at the mouth of Delaware bay; and to the northward as far as the

* See Leaming and Spicer, "Grants and Concessions of New Jersey," p. 8 *et seq.*

northermost branch of the said bay or river of Delaware, which is forty-one degrees and forty minutes of latitude, and crosseth over thence in a strait line to Hudson's river in forty-one degrees of latitude."

The famous quintipartite deed which defines the division between East and West Jersey was executed in July, 1676. This was preceded by some other conveyances which it is not necessary to consider. The southeasterly terminus of the partition, or province line, is described in this quintipartite deed as "the most southardly point of the east side of Little Egg Harbour aforesaid;" but Little Egg Harbor Inlet has shifted over several miles in location during the intervening centuries.

The Proprietors, who derived their title in the manner just outlined, sold parcels to applicants. Briefly, the procedure was to send out a deputy surveyor who located the lands applied for and then made his return to the Proprietors. Much of this survey work of the Eighteenth Century and the early part of the Nineteenth Century was very inaccurate, as might be expected in view of the very low value of the lands.

Earlier Surveys

From relatively early times, the State of New Jersey did show a commendable desire to obtain good maps and probably was pre-eminent among the States of the early Nineteenth Century in endeavoring to obtain them, but all that was accomplished still left much to be desired. The first real system of surveys, worthy of the name, was that established by the United States Coast and Geodetic Survey in about 1840. To-day this furnishes the chief reliance in defining the position of shore lines before the more detailed surveys, made necessary by the development of the coastal cities. It must be remembered here that the magnificent cities of the New Jersey Coast are institutions dating back very few years. Let this great evolution serve as an example to people of other localities, who may believe that their beaches will not develop into valuable lands in the near future.

The State Geological Survey of New Jersey made a valuable contribution in producing the surveys which form the basis for the maps now published by the Board of Conservation and Development. The topographical survey of that Department is reviewed in the report of the State Geologist of 1887, which recommended that the entire State be covered by a cadastral and economic survey. It consisted essentially, as to the coastal region, in transit and stadia surveys of the marshes and waterways, and was controlled by the U. S. Coast and Geodetic Survey monuments and triangulation stations. Unfortunately, the Legislature did not carry out the recommendation of the report.

Modern Water-Front Surveys

The Board of Commerce and Navigation (successor to the Riparian Commission), the Department of Inland Waterways, the Harbor Commission, etc., are extending their surveys over the water-front of the State as rapidly as possible. They require surveys for such different purposes as: The establishment of riparian or pierhead and bulkhead lines; the improvement of inland waterway channels; and the measuring of rate of erosion or accretion on the beaches. As far as practicable, each survey is planned to meet the needs of all

Departments. It is hoped that within a very few years the Board will have accurate detailed surveys of the entire tidal water-front of the State. This information is of untold value to land owners. It will remove from future litigation, the element of uncertainty in the location of high-water mark boundaries.

Most maps of the former Riparian Commission of New Jersey are very valuable, but they necessarily cover rather limited areas because they were made merely to meet the demand for the establishment of riparian lines. The former Department of Inland Waterways which constructed the inland waterway system from Cape May to Bayhead, covered some of this area with surveys of high precision.

Lack of Accurate Surveys

The legal complexities which arise through a confusing description of a beginning point are most serious and discouraging. A most striking example is the mischief caused by uncertainty in the location of the Province Line between East and West Jersey. The line was run by George Keith, Surveyor General of East Jersey, in 1687; but it has been stated* that, "the western proprietors thought too much of their best lands were surveyed to the eastward, and were uneasy with it." In Governor Coxe's report (1687) to the Proprietors of West Jersey, he compares the excellent maps of his adversaries of East Jersey with the unsatisfactory maps of his associates of West Jersey. He states,†

"They * * * of East Jersey have in this respect, exercised the highest prudence, knowing the whole country to a little, and thereby have both overreached you. I have seen their draughts than which nothing can be more exact;"

A map, dated 1747, shows another location of this line as run by Lawrence in 1743.

New Inlet, at the southerly end of Long Beach, opened in about 1800, during a violent storm. The position of Little Egg Harbor Inlet affects vitally the division line between properties. It entered prominently into the establishment of the county lines. In fixing the county line about 1885, it was found necessary to rely almost entirely on tradition to establish the time that New Inlet was opened because Beach Haven Inlet was not then in existence. A large number of the elderly people gave their ancestors' statements as evidence of its time of opening and its position at that time.

Comparison of Two Major Inlets

Thus, there are now two inlets: What is known as New Inlet, which opened in about 1800, and Beach Haven Inlet, farther north, which opened in 1920. A survey of Beach Haven Inlet was made in 1923 by the U. S. Coast and Geodetic Survey and the New Jersey State Board of Commerce and Navigation. Whether the two inlets will co-exist for any length of time is a question for the future. In the meantime the submerged lands have practically no value, and the isolated area known as Tucker's Beach has very little value. The behavior of this inlet, or pair of inlets, is in some degree paralleled by

* Smith's "History of New Jersey," p. 196.

† *Loc. cit.*, p. 546 *et seq.*

the behavior of Great Egg Harbor Inlet between Longport and Ocean City, which has, at times, had two branches separated by a large sand-bar island. The Lake Survey of 1881 shows a single inlet, with Longport Point far south of its present position. Longport has since lost approximately 184 acres (Fig. 1).

Example of Accretion

Sand beaches tend to shift oceanward or landward in the course of years. The tendency of sand dunes to move is well known and the existence of a series of lines of sand dunes indicates movement of the beach line. The late Henry S. Haines, M. Am. Soc. C. E., in fixing a survey on an undeveloped section of one New Jersey beach, drove iron pipes which extended 3 or 4 ft. above the ground level. They were set at the foot of the sand dunes on the ocean side. Some years later, when called upon to retrace this survey, he found the beginning corner almost buried and on the opposite, that is, landward, side of the sand dune. In other words, the sand dune had moved oceanward perhaps 150 ft.; yet this change was not apparent to the eye. Most beaches change slowly at points remote from the inlets; but at the inlets serious changes may be caused by one storm, if coast-protection measures have not been provided.

Survey Methods

The State has received about $12 000 000 from the sales and leases of lands under water and has expended about $2 000 000 in the improvement of its inland waters and protection of its beaches. The surveys of the New Jersey Board of Commerce and Navigation should be made for three purposes in order to be of maximum value; namely, the establishment of riparian grant lines; the possible improvement of the waterways; and, the measurement of the rate of change in the shore line. By shore line is meant the line of mean high water, for that is a property division line in New Jersey. This necessitates somewhat more detail than would be required for navigation maps and the accuracy should approximate 1 in 10 000. Traverse lines, carefully referenced to existing curb lines, board walks, and street monuments, are used in and near cities on the beach front. Frequent cross-sections of the beach are taken out to low water or beyond, as the mere running of a traverse along the high-water mark of a certain date does not give all the information desired. Where visibility permits, triangulation systems are established, the details usually being filled in by transit and stadia. The sextant and range method of sounding is used in the inlets; and the range and two-transit method in the bays and creeks. Wherever possible a chain of triangles or quadrilaterals forms the control, with distant point angles for references. Check bases are always measured in the triangulation systems.

Suggested Co-Ordination of State and Federal Surveys

An effort is now being made to persuade the U. S. Coast and Geodetic Survey to re-establish its precise triangulation system and leveling net over the New Jersey shore. The State has offered to adopt the Coast Survey standards and methods of observation for the extension of the triangulation and leveling. This plan would furnish an ideal medium of co-operation be-

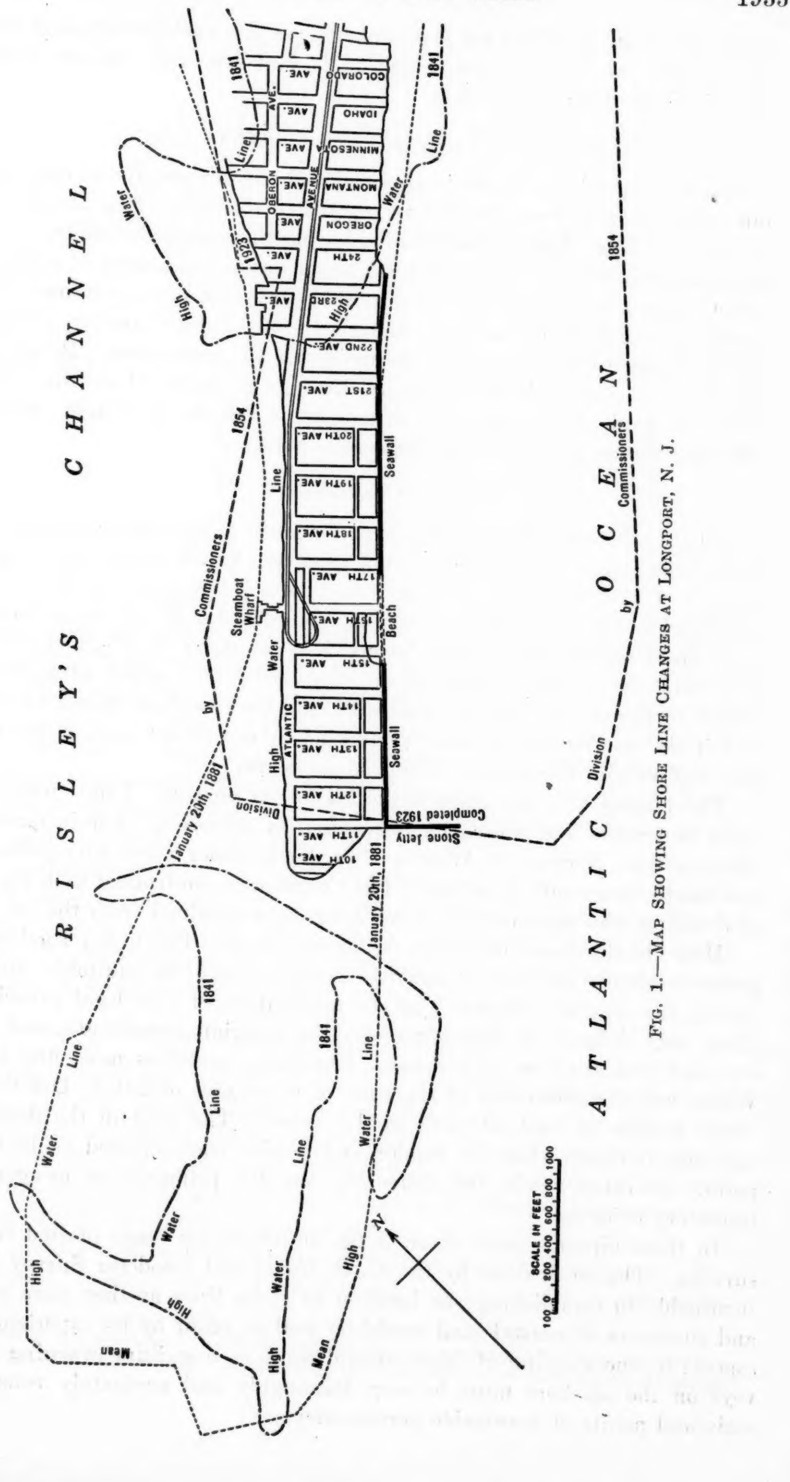


FIG. 1.—MAP SHOWING SHORE LINE CHANGES AT LONGPORT, N. J.

tween the State and Federal Departments, leaving each Department to devote its energies to the field for which it is best adapted; thereby tending to extend the existing survey data.

Importance of Control

Strong control of surveys is essential and may frequently be obtained without excessive cost over the ordinary traverse methods. The history of coast surveys in New Jersey teaches the lesson that accurate control and good reference points are of the most vital importance. No survey is worthy of the name unless it can be accurately retraced. Precision of measurement is only a means of securing consistent comparisons with other similar surveys, particularly adjoining surveys and re-surveys of the same tract. By proper control it is often possible to save much time in execution of details. Fine precision of angle or distance measurements is frequently sought without any adequate means of preserving the results obtained.

Conclusion

What is the need of a fine degree precision in angle and linear measurement if the high quality of execution is not preserved by adequate supporting monuments? Ultimately, the lands in question will be re-surveyed, whether for sub-division, partition, or other reason. Marsh islands and waste dunes of 15 or 20 years ago have been transformed into beautiful resorts, and this process will certainly continue. Serious questions arise with rapid advances in land values. Theoretically, the original work and the re-survey should be consistent and if the two operations were performed by competent men, using the same axes and origin, the results should be the same.

The history of shore lines is the history of change. Lands may be swept away by erosion and subsequently restored by accretion. For instance, in the Absecon Inlet Section of Atlantic City, lands under water fifty years ago are now worth many million dollars. The Courts were confronted with the problem of deciding who was entitled to the lands thus regained from the sea.

How should these accretions be apportioned? The broad legal principle, perfectly simple and just, is that the accretion shall be equitably admeasured among the riparian owners. In the application of this legal principle it is often very difficult to determine who the riparian owners are, and how the regained land shall be apportioned. Important questions must first be solved. Where was the shore line at the time of a transfer of title? Did the vendor retain a strip of land adjacent to the water? The calls in the deed may or may not indicate what the vendor and vendee contemplated as to this vital point. In other words, the difficulties are due primarily to meager or contradictory evidence.

In these circumstances there is no substitute for maps plotted from good surveys. The work done by the U. S. Coast and Geodetic Survey has been invaluable in establishing the location of shore lines as they were years ago, and surveyors of coastal land would do well to profit by its experience. Most especially, the shifting of inlets should serve as a striking warning that surveys on the seashore must be very thoroughly and accurately referenced to mainland points of reasonable permanency.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

ADVANCES IN WATERWAYS ENGINEERING DURING A HALF CENTURY*

W. M. Black,† M. Am. Soc. C. E.

Synopsis

A review of the progress of waterways engineering must include statements of advances made both in the science and in the art of engineering. Since an engineering problem is solved best when the desired results are obtained in the least time and at the least cost, progress in engineering is measured by the improvement of methods and the reduction of costs. The art of engineering is very old. It is questionable whether more noteworthy examples will ever be produced than those exemplified in the pyramids and temples of Egypt, the irrigation works known to ancient peoples, the aqueducts of Rome, or the walls and canals of China.

Modern engineering can boast of great triumphs, but the advances made have been mainly in plant and methods causing reductions of time and costs. In this paper, the writer will not be able to treat this subject from the world-wide viewpoint which it deserves. He will simply try to show some of the progress made in waterways engineering in the United States.

Science of Waterways Engineering

Researches into the laws governing the flow of water in streams date back through the centuries, but it is hardly too much to claim for Humphreys and Abbot that present knowledge of stream flow is largely based on their pioneer work in the Mississippi, carried on during the middle of the Nineteenth Century. Since then, engineers of both Europe and America have continued the research until, to-day, formulas are available by means of which the discharges of streams can be determined with a fair degree of accuracy. The relation between rainfall and stream discharge is yet under investigation.

Note.—Written discussion on this paper will be closed in **February, 1928.**

* Presented at the meeting of the Waterway Division, Philadelphia, Pa., October 8, 1926.

† Maj.-Gen., U. S. A. (*Retired*); Cons. Engr. (Black, McKenney & Stewart), Washington, D. C.

Many data have been collected and tentative formulas proposed. The complexities which this subject presents are well shown in papers published by the Society.

The question of flood control has also received wide consideration by American engineers and has been the subject of papers and discussions before the Society. The Assouan Dam, in Egypt, and the work of the Miami Conservancy Commission, in Ohio, are notable examples of works constructed to this end during the past fifty years. The relation of deforestation to river floods has been widely discussed, notably by Chittenden,* Burr,† and Townsend.‡

The action of side channel spillways has been well discussed§ by Julian Hinds, M. Am. Soc. C. E. The efficacy of such spillways in controlling flood heights is under trial in the Sacramento River, California, and is being studied for the Mississippi River by a Board of Engineers.

The effects of a regulated flow from storage reservoirs in maintaining navigable depths in the Mississippi River above Lake Pepin, and in decreasing flood heights, have been under observation and record since 1880, when the reservoir project was authorized, affording a material increase in exact data on this important branch of waterways engineering.

The volumes, movement, and heights of floods in the Mississippi River have received prolonged study by the members of the Mississippi River Commission and their assistants, and to-day it is possible to predict, for points on the lower river, the height which will be attained by a flood wave passing St. Louis, Mo., or Cairo, Ill. The phenomenon of the increase of the velocity of flood wave propagation over that of flood river current, slight in shoal water, but great in deep reaches, has been studied and announced. Another important addition to waterways engineering knowledge, is the better understanding of the courses followed by the filaments of water of a river in their progress through a bend, and the better understanding of the processes of bank erosion and bar formation obtained therefrom. The important functions of bends in a river which flows through an alluvial plain are also now better appreciated. "Boils" in the deep water of bends and the cause of the formation of the "mud lumps" near the mouth of a river like the Mississippi, are yet under study. The general principles of waterways engineering as now generally accepted, have been ably outlined by Townsend.‖ Progress has been made in the study of tidal action in rivers and estuaries and in canals which join two bodies of water in which tidal action exists. Parson's study of the tides and their action in the Cape Cod Canal,¶ Marmer's studies of the tides and their action in

* "Forests and Reservoirs in Their Relation to Stream Flow, with Particular Reference to Navigable Rivers," by H. M. Chittenden, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXII (1909), p. 245.

† H. R. Doc. 9, 62d Cong., 1st Session.

‡ *The Military Engineer*, Vol. XIV, 1924, p. 242.

§ *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 881.

‖ "The Hydraulic Principles Governing River and Harbor Construction," C. McD. Townsend, M. Am. Soc. C. E., The MacMillan Co., 1922.

¶ "The Cape Cod Canal," by William Barclay Parsons, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1.

New York Harbor,* and the report of Zeskind and Le Lacheur on those of Delaware Bay and River,† give valuable data. The best study of this subject known to the writer is that of L. Bonnet, Engineer and Director of Ponts et Chaussées, Belgium, on tidal action in the Scheldt River,‡ in which the magnitudes of the action of the various tidal phenomena in any reach of the river are shown to be functions of the tidal energy entering that reach, and formulas are given by which, for any proposed change in the channel capacity of the reach, the corresponding change in the tidal phenomena can be predicted within reasonable limits of accuracy. This subject is now being studied on the Chesapeake and Delaware Canal.

The great developments of the past half century in irrigation and in hydro-electric production have led to increased knowledge of the principles governing works constructed for these purposes, but these subjects are beyond the scope of this paper.

The formation of an up-stream undercurrent of salt water at and near the mouths of the Mississippi River, when the main river current is not reversed in direction, was first investigated by Lt. Col. E. J. Dent, Corps of Engineers, U. S. A., M. Am. Soc. C. E., and described§ by Col. C. McD. Townsend, U. S. A. (*Retired*), M. Am. Soc. C. E. The subject of bottom salt-water undercurrents was also studied in 1921-22 by Capt. (now Major) A. P. von Deesten, Corps of Engineers, U. S. A., Assoc. M. Am. Soc. C. E., in the Sabine-Neeches Waterway, Texas.‖ The action of a lock between an upper fresh-water pool and a lower pool of salt water, in pumping salt water into the upper pool, has been studied at the Charles River Lock, Boston, Mass., and the Gatun Locks of the Panama Canal. There have also been made many studies of the effects of storm wave and current action on sandy coasts.¶

The causes and effects of upward water pressure on dam foundations and the measures necessary to overcome this pressure have been widely investigated. Many studies have been made of the stresses in arches and particularly in the arch of a single arch dam, under the supported pressures.

All these studies form additions to the science of waterways engineering.

Art of Waterways Engineering

The past fifty years have produced notable advances, largely due to increased knowledge of the properties of materials, to the use of electricity for power and light, to the development of steam and internal combustion engines, and to the great improvement of machines thereby made possible. Although concrete in one form or another has been in use for centuries, its wide application to all classes of structures is less than fifty years old, and

* "Tides and Currents in New York Harbor," H. S. Marmer, *Serial No. 285*, Dept. of Commerce, U. S. Coast and Geodetic Survey.

† *Serial No. 336*, Dept. of Commerce, U. S. Coast and Geodetic Survey.

‡ "Contribution à l'Etude Théorique des Fleuves à Marées et Application aux Rivieres à Marées," par L. Bonnet, Ingenieur en Chef, Directeur des Ponts et Chaussées, *Annales* des Travaux Publics de Belgique, Fascicles 3, 4, 5, et 6, 1922, et 1, 2, 3, et 5, 1923.

§ *The Military Engineer*, Vol. XVIII, (1926), p. 99.

‖ H. R. Doc. No. 12, 64th Cong. 2d Session, and *The Military Engineer*, Vol. XV, No. 84 (1923), p. 527.

¶ *Transactions*, Am. Soc. C. E., Vol. XXIII (1890), p. 123; Vol. XXIX (1893), p. 223; Vol. L (1903), p. 66; Vol. LXXX (1916), p. 1786; and Vol. LXXXVII (1924), p. 589.

this wide use is largely due to the employment of steel in reinforcement. As a result of this use, the properties of concrete and the action of the reinforcement have received extended study, with the development of numerous formulas for practical application. A recent advance has been in the development of alumina cements which attain their strength in a short time after setting.

Hydrographic Surveying.—Aeroplane photography has greatly facilitated the work of hydrographic surveying. By its aid, extended surveys can now be made periodically with relatively small expenditures of time and money, and changes in the course of a stream, or in a coast line, noted and their cause studied.

The use of drags or sweeps in determining navigable depths and discovering submerged obstructions has become general and the sweeps themselves have been improved in form.

The first sweep designed to cover large areas rapidly was devised by Capt. S. S. Leach, Corps of Engineers, U. S. A., and used by the U. S. Lake Survey in 1893-95.* Subsequently, in 1902, it was greatly improved by F. C. Shenehon, M. Am. Soc. C. E., then Principal Assistant Engineer of the Lake Survey.† As developed, the sweep can cover a swath 2 000 ft. wide. In 1908, the maximum day's work of one sweep covered 5 sq. miles.‡ The use of the sweep was adopted by the U. S. Coast and Geodetic Survey about 1904. In a recent survey in Alaska, the Coast and Geodetic Survey party was able to cover 65 sq. miles in one day's work.§ The construction and operation of the wire drag and sweep are described in recent publications of the U. S. Coast and Geodetic Survey:‖

A method of making continuous lines of soundings was developed by the U. S. Lake Survey in 1899-1900, by the use of a heavy weight suspended by a fine wire from a drum, the revolutions of which gave the depth. The weight was raised just clear of the bottom after each sounding. Soundings were made at intervals of 20 sec.¶ A few years afterward the late E. S. Wheeler, M. Am. Soc. C. E., invented an instrument called a bathometer by which a continuous profile of the bottom can be delineated on paper.**

The most important improvement of sounding apparatus is one quite recently developed by the Engineer Department of the U. S. Navy, called the "Sonic depth-finder." With this instrument deep-water depths can be determined at short intervals without stopping the course of a vessel. This instrument has proved of special value in work connected with the placing and repair of submarine cables in ocean depths. With this instrument the Hydrographic Office, U. S. Navy, is making bathometric charts of selected ocean areas. In "Bathometric Chart No. 5194, San Francisco to Pt. Descanso", the ocean bed was covered between the 100 and 2 000-fathom contours,

* Rept., U. S. Chf. of Engrs., 1896, p. 4063.

† *Loc. cit.*, 1903, p. 2763.

‡ *Loc. cit.*, 1909, p. 2496.

§ "The Evolution of the Nautical Chart," by E. Lester Jones, M. Am. Soc. C. E., *The Military Engr.*, Vol. XVI, 1924, p. 219.

‖ U. S. Coast and Geodetic Survey, *Special Publication No. 118.*

¶ Rept., U. S. Chf. of Engrs., 1902, p. 2781.

** *Loc. cit.*, 1908, p. 2528.

with soundings in parallel courses, 10 or 5 miles apart. In 38 days of field work 34 000 sq. miles were contoured by 5 000 soundings made at a ship speed of 12 knots. In working up the data collected by the field parties, a new instrument called the "Pantograver" has proved of value. With attachments eliminating distortion of cloth or paper, it is hoped eventually to be able to engrave plates from photographs; one such attachment is now in use, but it concerns one direction only.*

Many improvements in instruments and methods have been made by the U. S. Coast and Geodetic Survey. Recently, a portable automatic tide gauge has been developed by G. T. Rude, Commander, U. S. Coast and Geodetic Survey, M. Am. Soc. C. E.,† which can be easily carried and installed in the field. A current direction indicator has also been devised with which current directions can be determined simultaneously at three different depths on the same vertical.†

The U. S. Coast and Geodetic Survey has developed a method whereby the position of a survey ship can be accurately determined when it is beyond the range of visibility of shore objects and when closer to shore during low visibility on account of fog and haze. The method also permits surveying during the hours of darkness with the result that offshore hydrography has been materially speeded up by permitting the ship to work continuously throughout the day, and even on those days when formerly field work was impossible. This method and the apparatus are fully described.‡

The Bureau has conducted extensive tests on various types of echo sounding apparatus, including the Navy sonic depth finder and an apparatus invented by the Submarine Signal Corporation known as the Fathometer. The Fathometer was designed primarily as a navigational sounding machine, and, in its original form, did not possess quite the accuracy required for surveying. However, this defect has been eliminated by the addition of a suitable governor on the timing apparatus so that now results can be obtained with the Fathometer quite as accurately as by any echo sounding apparatus and with almost the precision of direct wire measurements. The Bureau has two ships equipped with this apparatus and expects to so equip all surveying ships. The particular merit in this type of echo sounding apparatus lies in its ability to measure moderate and shallow depths. The Fathometer to date gives very accurate results down to about 15 fathoms, and there is reason to believe that with certain definite improvements the range may be extended to perhaps 4 or 5 fathoms.

Two sounding machines of the reel and wire type have been developed by the Bureau, one for use in great depths and the other for depths of 400 to 500 fathoms. These machines consist of reels driven by electric motors and controlled in such a way that the reels may be stopped instantly and started

* *The Military Engineer,* Vol. XVII (1925), p. 39.

† "Tide and Current Investigations of the Coast and Geodetic Survey", by E. Lester Jones, M. Am. Soc. C. E. (In press); also, "Tides and Their Engineering Aspects," by G. T. Rude, M. Am. Soc. C. E., *Proceedings,* Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1092.

‡ "Radio Acoustic Method of Position Finding in Hydrographic Surveys," by Hech, Eckhardt, and Keiser, *Serial 227,* Dept. of Commerce; U. S. Coast and Geodetic Survey, *Special Publication No. 107.*

at any desired speed. While no new principles are involved in this apparatus, it is, nevertheless, a great improvement over the older type, due to its simplicity, sensitiveness, and flexibility, all of which result in greater speed of operation and greatly reduce the danger to the submerged apparatus, detaching rods, water specimen bottles, and thermometers. It has been found that inexperienced personnel can operate these new machines as efficiently as trained operators could handle the older types. As an illustration of this point, it may be mentioned that one survey ship with a new and untrained crew carried a line of soundings from New York to San Diego and, in addition, explored the deep north of Porto Rico without once parting the wire or losing any equipment. That, it is believed, established a new record for oceanographic work.

Pressure tubes for measuring depths as great as 100 fathoms with considerably greater precision than the commercial glass tubes have been developed in this Bureau and are now used extensively on surveys where heretofore such apparatus could not be tolerated because of its unreliability. These tubes record depths with a probable error not in excess of 5% and are now used extensively for depths greater than 15 fathoms.

Dredging.—Dredges operated by steam power have been used in river regulation for more than a century, but during the past 50 years their efficiency has been vastly increased. In addition to the old type clam-shell, dipper, and ladder dredges (all of which have been greatly improved), hydraulic pump dredges have come into general use.

These dredges are of two general classes: Hopper dredges and pipe-line dredges. The hopper dredge, while under way, excavates material by means of a drag attached to the end of a suction pipe of a pump and discharges into hoppers in the ship. It was invented by a Mr. Lebby, at Charleston, S. C., in 1855, where the first of its type was used in 1857. After the Civil War a similar dredge was built in 1871, by the late Brig.-Gen. William Ludlow, U. S. A., M. Am. Soc. C. E., (then Captain, Corps of Engineers, U. S. A.), for use on the South Atlantic Coast. It is the only type of dredge which can be used efficiently in a seaway and is now in general use throughout the world for deepening the entrances to harbors and rivers, having been markedly developed in efficiency and power. By its aid, the problem of securing and maintaining entrance channels protected by jetties has been greatly simplified. In the latest dredges of this type, the propelling, steering, and pumping machinery is actuated by electricity generated from power supplied by internal combustion engines.

These electric dredges, due to their ease of handling and close control, are capable of being operated easily in a very limited space, such as in narrow channels, or within a very short distance of piers, which would be both difficult and more dangerous with steam dredges. While cost records for a complete year of operation are as yet unavailable, it is estimated that operating expenses will be 20% less than those of the same type of steam dredge. Maintenance expenses can as yet only be estimated.

These dredges have been largely used on experimental work, with a view to determining the type of dredging for which they were best fitted, resulting naturally in an increased cost of operation. On certain classes of dredging the cost has been as low as 3 cents per yd., with approximately 400 000 cu. yd. removed monthly.

Pipe-line hydraulic dredges were first used on the Pacific Coast. These swing in a position which is slowly moved forward, excavate material by means of a revolving cutter at the bow, just in advance of the end of a suction pipe, and discharge through a pipe line supported on floats.

Fig. 1 is a view of the U. S. Pipe-Line Dredge *Wahalak,* showing an old type cutter for soft materials. Fig. 2 shows a new type cutter for soft materials on the U. S. Pipe-Line Dredge *C. B. Harris.*

This type has also received a great development. While the general radius of delivery through the pipe line had averaged about 2 500 ft., with an elevation not exceeding 10 ft., recently, in the Chesapeake and Delaware Canal, a dredge of this type delivered material at a distance of more than 500 ft. and at an elevation of more than 90 ft. In the Panama Canal, another pipeline dredge, aided by a relay pump on a barge, delivered through a pipe line, 1 500 ft. long, to an elevation of 250 ft., or through a pipe line, 15 840 ft. long, to an elevation of not to exceed 20 ft. This work was done at an average cost of 28 cents per cu. yd.*

It is claimed that a late type of this class of dredge, working in Florida with a 16-in. pump, excavated sand and mud, and delivered at a distance of 3 000 ft. at a cost of 7 cents per cu. yd., and when equipped with a special cutter head excavated and removed the limestone rock found there to a depth of 22 ft. at a cost of 40 cents per cu. yd. Fig. 3 shows the rock cutter on the U. S. Pipe-Line Dredge *Barnard.*

On the Mississippi River, a special class of this type of dredge has been developed for the removal of the bars of fine material found there. In this type, no revolving cutter is used, the material being stirred up by water jets in advance of the suction pipe.

Most of these Mississippi type dredges are self-propelled and resemble in outward appearance a stern-wheel, shallow-draft Mississippi River steamboat. On the hull are mounted power units; a centrifugal dredging pump; a wide structural ladder hinged in a well recess at the bow of the dredge and carrying at its lower end a dustpan-shaped suction head 20 ft. or more wide, which is provided with water-jet nozzles to loosen the material; a hauling winch; hoisting winches; spuds; the necessary auxiliaries and controls for operation; and suction and discharge piping and floating pipe line.

In operation, the dredge steams to a point above the bar, drops two anchors one on each side of the channel to be cut, then drops several hundred feet down stream on the anchor cables, to the lower edge of the bar. The dredge pump, and the pump furnishing water for the jet agitator, are started, the dustpan suction head is lowered to the bottom, and the dredge is hauled up stream by the hauling winches, with the suction head against

* J. G. Claybourne, *The Military Engineer,* Vol. XVII (1925), p. 910.

the bar. The discharge is from the side of the dredge and through a short floating pipe line, 500 to 1 000 ft. long, mounted on pontoons. The pipe line is held in a line abreast of the dredge by the dynamic action of the stream on a baffle-plate located at the end of the line. The dredge material is deposited in the stream, but along the edge of the channel, where it forms a dike which assists in deepening the channel by the scour produced by its action in regulating the direction of the stream.

When the upper edge of the bar is reached the dredge drops back again and makes similar cuts alongside of and parallel to the first cut. Thus, a channel is cut through the bar, and this channel is later widened and deepened by the action of the current through it.

The cost of dredging varies with the relative number of hours of work and of idle time, with efficiency of the plant, the depth of dredging, the character of the materials to be excavated, and the location at which spoil is deposited. For each type of dredge there is a class of excavation for which that type is best fitted, and no judgment can be formed of the relative efficiency of the types without a full knowledge of the conditions found in the work.

In this connection the record of the recent performances of hydraulic dredges used on the waterways engineering works of the United States under the charge of the Engineer Department, U. S. Army, is of value. This record, Tables 1, 2, and 3, includes dredges which worked for the entire year.

Rock Excavation.—As stated previously, soft rock in place can now be excavated by the hydraulic dredge when equipped with a special toothed cutter. Rock in thin strata can be broken by the chisel dredge, first used in the Mississippi River in 1855, but much improved since. The Lobnitz rock cutter is only a development of the old type.

Drill boats, generally equipped with piston and well drills, for drilling and blasting rock under water, also have been markedly improved. Hollow pipe drills have come into use since devised by Col. George McC. Derby, Corps of Engineers, U. S. A., M. Am. Soc. C. E. (then Lieutenant Corps of Engineers, U. S. A.), in 1883, in the excavation of Flood Rock, Hell Gate. There, too, the simultaneous explosion of 44 175 charges of rack-a-rock placed in the drill holes in the walls of the galleries cut in Flood Rock, was effected by the method of sympathetic explosions, by the explosion of charges of dynamite laid along the center lines of the galleries. These gallery charges were fitted with fuses which were connected with a firing battery.*

The improvements in drilling machinery, in electric firing batteries and exploders, and in explosives, have greatly facilitated and cheapened this class of work.

River Regulation.—The advances made in the methods of regulation works can be appreciated from a comparison of the methods used by the late Brig.-Gen. O. H. Ernst, U. S. A., M. Am. Soc. C. E. (then Captain, Corps of Engineers, U. S. A.),† in 1879, at Horsetail Bar, Mississippi River, near St. Louis, Mo., with those now in use on the Mississippi, as described by

* Rept., U. S. Chf. of Engrs., 1886, p. 682.

† *Loc. cit.*, 1880, Pt. 2, p. 1362.

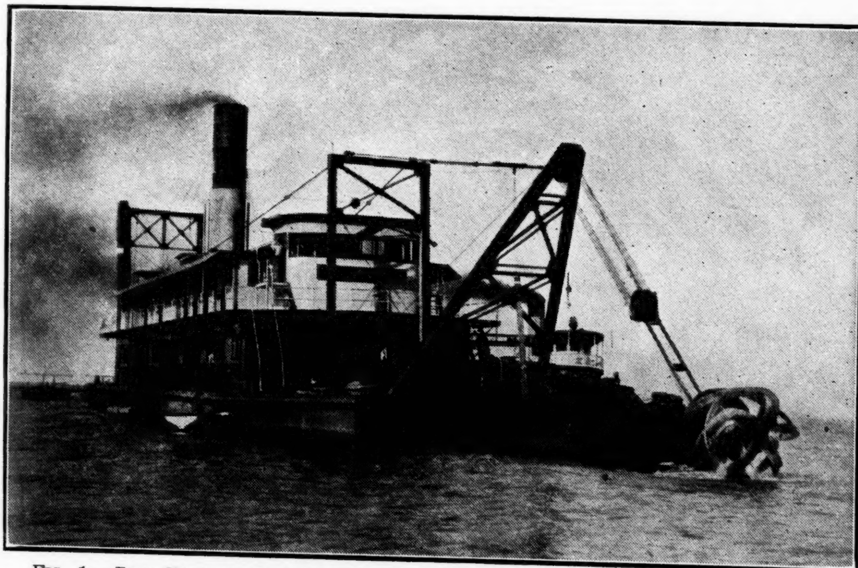


FIG. 1.—BOW VIEW OF U. S. DREDGE *Wahalak*, SHOWING OLD TYPE CUTTER FOR SOFT MATERIALS.

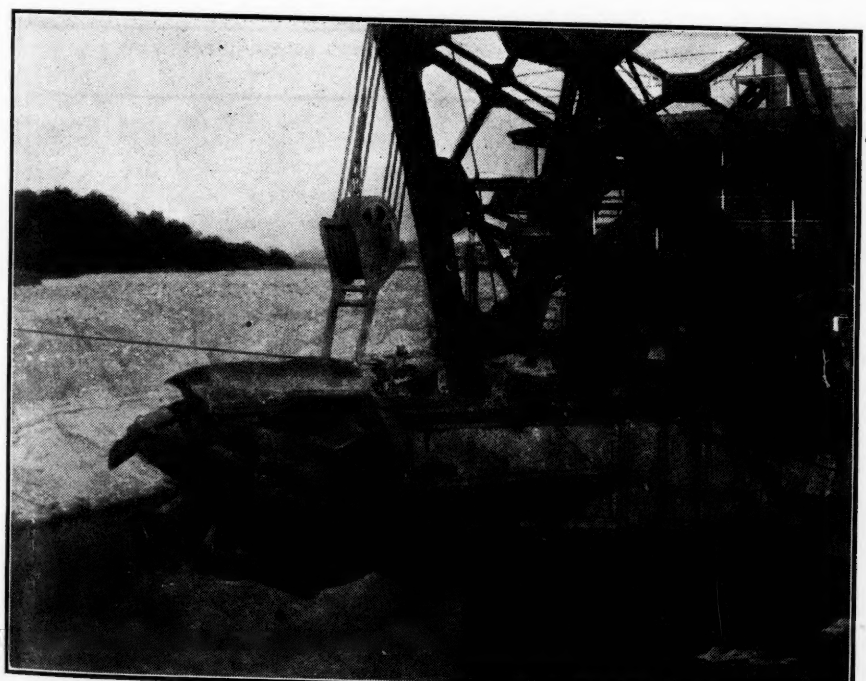


FIG. 2.—NEW TYPE CUTTER, U. S. PIPE-LINE DREDGE *C. B. Harris*.

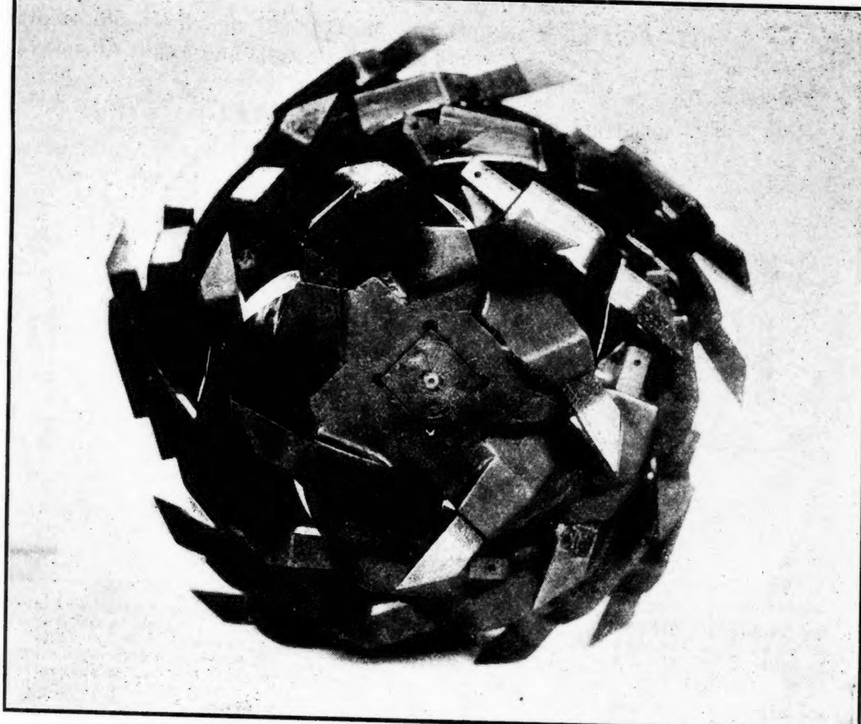


FIG. 3.—ROCK CUTTER, U. S. PIPE-LINE DREDGE *Barnard*.

FIG. 4.—TYPE OF BRUSH AND POLE MATTRESS.

Jones and Skelly, in 1921,* Todd and O'Connor, in 1923,† Howell, in 1924,‡ Potter, in 1925,§ and Gotwals, in 1926.‖

TABLE 1.—SEA-GOING HOPPER DREDGES.

1923.		1924.		1925.	
No.	Size of pump, in inches.	No.	Size of pump, in inches.	No.	Size of pump, in inches.
6	15	5	15	5	15
2	16	2	16	2	16
2	17	1	17	1	17
4	19	4	19	4	19
4	20	5	20	5	20
2	26	2	26	3	26
1	30	1	30	..	..
21	..	20	..	20	..

AVERAGE PERFORMANCES.

Item.	1923.	1924.	1925.
Total number of cubic yards dredged	1 727 230	2 071 741	2 197 796
Total hours at work.*	4 185.52	4 664.84	4 756.13
Total hours lost from work †	4 574.48	4 119.18	4 003.87
Percentage of time at work*	47.7	53.1	54.3
Percentage of time lost from work	52.3	46.9	45.7
Total field operating cost	$130 451	$148 815	$140 172
Total cost of repairs	$ 31 828	$ 52 815	$ 40 508
Gross operating and repair cost	$162 279	$201 630	$180 680
Total cost, in cents per cubic yard	9.25	9.73	8.22

* Time at work includes: Pumping; turning in cut; to and from dump and dumping; taking on fuel and supplies; to and from berth; and miscellaneous.

† Time lost from work includes all other time.

Prior to 1878, the guiding and cross-dikes were built solidly of stone and brush, placed on the river bottom. When this bottom was soft and erodible, these dikes were quickly undermined and destroyed. Under the direction of Captain Ernst, use was made of brush mattresses (Fig. 4) for holding the bottom, and of permeable screens of brush and of Brownlow weeds for reducing current velocities and inducing fill. Although these proved effective, they were too fragile. Brush mattresses are still used, but of improved construction. The latest advance has been in building brush mattresses of fascines, or of substituting for the brush mattress, an articulated, reinforced concrete mat. Fig. 5 shows a type of woven mattress and Fig. 6 is a view of concrete bank revetment at Gasconade, Mo. The permeable dikes have been made stronger and more permanent in form by the use of pile bents firmly connected. Quite recently, reinforced concrete piles and caps have been used with good results in cost. Fig. 7 shows the results of the use of a retard (permeable dike)

* *The Military Engineer*, Vol. XIII, 1921, p. 197.

† *Loc. cit.*, Vol. XV, 1923, p. 215.

‡ *Loc. cit.*, Vol. XVI, 1924, p. 283.

§ *Engineering News-Record*, Vol. 94, 1925, p. 508.

‖ *The Military Engineer*, Vol. XVIII, 1926, p. 236.

on the Missouri River. The main channel was along the bank at the time of construction.

The system of "retards" made of trees, now extensively used for bank protection, is only a larger "Brownlow weed" firmly anchored by concrete piles sunk deeply in the river bottom by the water-jet process. The channel for the jet is cast in the center of the pile, with exits for the water distributed at intervals along the pile length.

TABLE 2.—Hydraulic Pipe-Line Dredges. Cutter Type.

1923.		1924.		1925.	
No.	Size of pump, in inches.	No.	Size of pump, in inches.	No.	Size of pump, in inches.
2	10	2	10	2	10
6	12	7	12	6	12
8	15	8	15	8	15
4	16	3	16	3	16
3	18	2	18	3	18
17	20	17	20	16	20
2	24	4	24	5	24
42	..	43	..	43	..

Average Performances.

Item.	1923.	1924.	1925.
Total number of cubic yards dredged	1 471 444	1 491 481	1 687 684
Total hours at work*	3 819.48	3 859.50	3 804.83
Total hours lost from work†	4 940.52	4 790.10	4 942.88
Percentage of time at work*	43.6	44.6	43.5
Percentage of time lost from work†	56.4	55.4	56.5
Total field operating cost	$ 94 687	$ 95 276	$ 94 661
Total cost of repairs	$ 27 710	$ 30 919	$ 30 877
Gross operating and repair cost	$122 397	$126 195	$125 538
Total cost in cents per cubic yard	8.32	8.48	7.44

Relative Cost per Cubic Yard for Different Sizes of Pump.

Cost per Cubic Yard, in Cents.

Size, in inches.	8.	10.	12.	15.	16.	17.	18.	20.	22.‡	24.
1923	0.226	0.195	0.139	0.209	0.164	0.070	0.092	0.067	0.376	0.060
1924	0.214	0.190	0.173	0.151	0.183	0.052	0.128	0.063	0.681	0.066
1925	0.219	0.190	0.160	0.108	0.128	0.116	0.122	0.065	0.150	0.045

* Time at work includes: Pumping; shifting and waiting on pipe line or scows; shifting dredge; passing vessels; going to and from berth; and miscellaneous.

† Time lost includes all other time.

‡ The higher costs for the 22-in. pump dredge were due to its use in excavating sand and coral rock.

The method of sinking piles by the aid of a water jet* was first used in Matagorda Bay, Texas, at the suggestion of the late Gen. George B. McClellan,

* "The Water Jet as an Aid to Engineering Construction," by the late L. G. Schermerhorn, M. Am. Soc. C. E., U. S. Asst. Engr., Engr. Dept., U. S. Army, 1881.

U. S. A. (then Lieutenant, Corps of Engineers, U. S. A.). It is said that he conceived the idea after watching sand crabs bury themselves. The water jet was used occasionally, thereafter, as an aid in pile-driving and in sinking caisson cylinders, but it was not until 1871 that its true function of relieving the pile from skin friction was established by Captain Brown, Corps of Engineers, U. S. A., in the operation of sinking screw-piles for the pier at Lewes, Del., when he found that the jet had to be applied above, and not below, the screw disk. The patented pile of the "retard" process takes full advantage of this principle. The use of the water jet in pile-driving is now almost universal. Reinforced concrete piles are also of recent origin.

TABLE 3.—Performance of Hydraulic Pipe-Line Dredges Working Under Contract.

1923.	
Total yardage	13 793 319
Total of contract prices	$2 824 197
Average price per yard, in cents	20.99
30 contracts 9 districts No rock included	
1924.	
Total yardage	25 016 843
Total of contract prices	$5 634 324
Average price per yard, in cents	21.3
29 contracts 8 districts No rock included	
1925.	
Total yardage	14 878 148
Total of contract prices	$2 970 327
Average price per yard, in cents	19.9
29 contracts 10 districts No rock included	

Levees.—In place of the old dump-wagon and horse-drawn scraper process, levees are now constructed more safely, more rapidly, and at less cost by the use of drag-line scrapers and of movable cableways from which scoop buckets are operated.* As a result of experience, the Mississippi River Commission has adopted a standard cross-section for levees, which is proving satisfactory.

Jetties.—Little change has been made in the methods of the construction of jetties at river and harbor entrances, except that brush mattresses are now rarely used, and then only as a foundation on an exceptionally soft bottom. For jetty construction, a trestle built along the jetty axis and carrying a railroad track is now generally preferred.

Canalization.—In canalization work various types of movable dams have been tried out and improvements made. By an improvement in the hurter

* *The Military Engineer*, Vol. XVI, 1924, p. 288.

(the cast-iron shoe against which the end of the prop rests when the wicket is up) of the Chanoine wicket dam, the necessity for the use of the troublesome tripping-bar can be avoided if so desired. A new type of wicket, invented by Guy B. Bebout, Assoc. M. Am. Soc. C. E., about 1913, was placed in service at Dam No. 13, Ohio River, in 1915. The advantages of this wicket are:

1.—It can be raised at any open-river stage to the top of wickets without serious loss of time.

2.—It can be raised against any difference in pool levels, sufficient power being applied at the chains.

3.—It can be lowered under all conditions of river stages and pool levels.

4.—When raising the wicket, drift cannot obstruct or become entangled in the parts to such an extent as to prevent this maneuver, except in very extreme cases, and in such cases the drift should be easily removed.

5.—A boat, barge, or heavy drift striking the top of the wicket will cause the wicket to collapse without damage to the boat or dam.

Bear-trap dams have been greatly improved and dams of this type are now in use with lengths deemed impracticable fifty years ago. Fig. 8 shows Lock and Chanoine Dam No. 24, Ohio River. The fixed wire shows in the foreground, with the bear-trap, partly down, just beyond.

Perhaps the greatest advances made in this branch of engineering art have been in the details of lock construction.

Coffer-Dams.—The invention of interlocking steel sheet-piling has made practicable the construction of coffer-dams of a new and simple type, exemplified in the coffer-dams for the Black Rock Lock, Niagara River, the coffer-dam for the *Maine,* Havana Harbor, the Browns Landing Lock, Cape Fear River, North Carolina, and that for the Forty-Sixth Street Pier, New York, N. Y. In this last,* designed by Charles W. Staniford, M. Am. Soc. C. E., the pressure head of water resisted was 65 ft.

Foundations.—This form of piling has also proved of great value for cut-off walls around and below lock foundations in permeable soils and for cut-off walls in earth dams.

Locks.—In 1892 and 1893, the late W. L. Marshall, Brig.-Gen. U. S. A. (then Captain, Corps of Engineers, U. S. A.), began the construction of the concrete locks of the Illinois and Mississippi Canal. These are believed to have been the first locks constructed entirely of concrete.† To-day, the use of stone masonry for lock walls is exceptional.

Gates.—In the lock of the Davis Island Dam (now removed) single-leaf gates, 118 ft. long, which when operated rolled on a track into or out of a masonry recess in the land wall, were used for the first time (1881-82).

Culverts.—The lock of the Davis Island Dam was filled through culverts in the side-walls, fitted with valves moved by hydraulic power. It was emptied through a culvert below the lower gate recess. In the locks of the St. Mary's Falls Canal of 1881 and, later, as in the Panama Canal locks, the filling and emptying culverts have their orifices in the floors of the locks.

* *Transactions,* Am. Soc. C. E., Vol. LXXXI (1917), pp. 498 and 553.

† Rept., U. S. Chf. of Engrs., 1894, p. 2164.

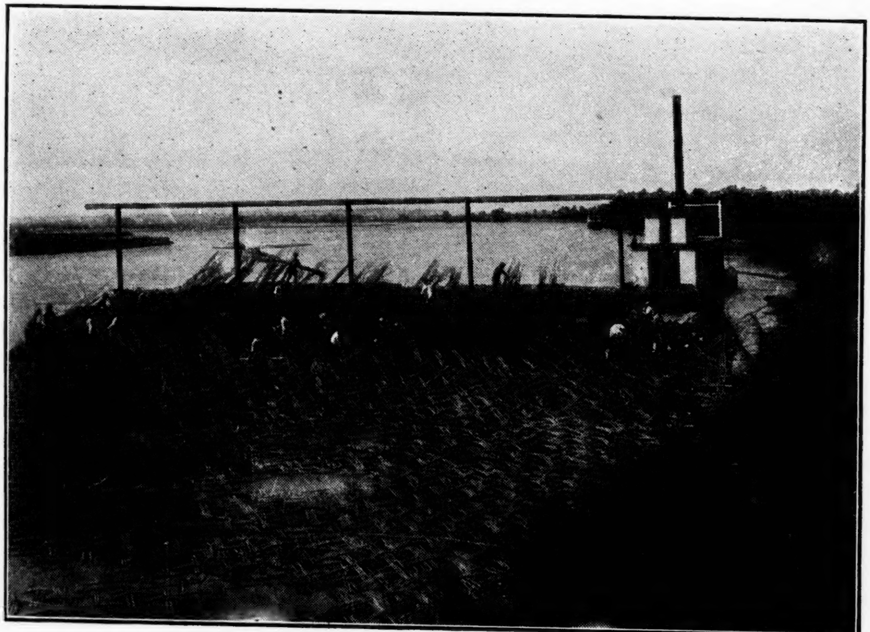


FIG. 5.—TYPE OF WOVEN MATTRESS.

FIG. 6.—VIEW OF GASCONADE BOAT YARD, SHOWING CONCRETE BANK REVETMENT.

FIG. 7.—BRUSH RETARD (PERMEABLE DIKE), SHOWING FILL RESULTING FROM (LOW-WATER) WINTER SEASON.

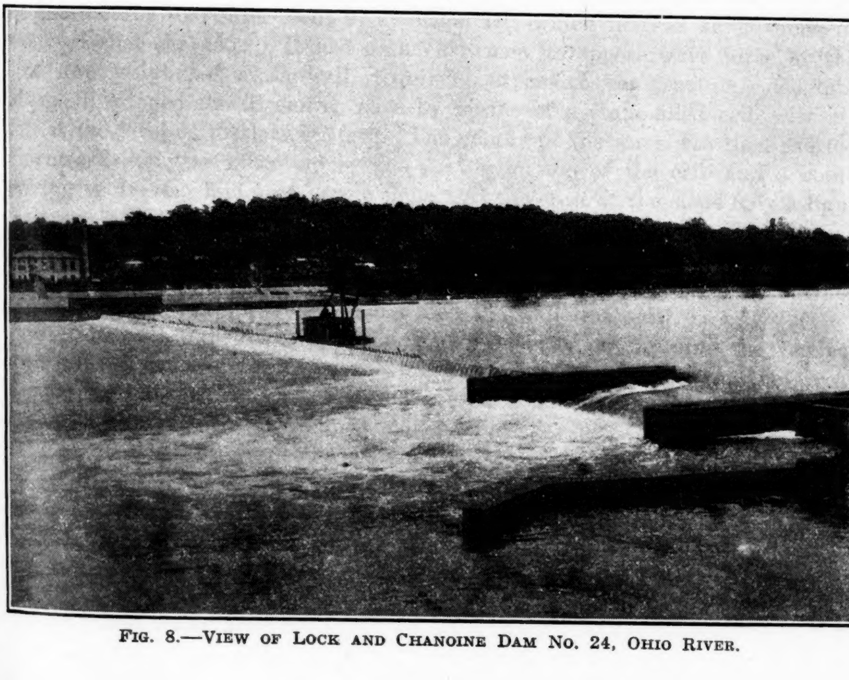


FIG. 8.—VIEW OF LOCK AND CHANOINE DAM NO. 24, OHIO RIVER.

Pap

eral
lock
slid
bein

imp
C.
gre

da
Th
de

thr
ow
loc

val

ha
Th
no

sh
pr
wo
si
M
to
do
pa
wa
fo
be
ly

w
L
a

b
c

f

I

Valves.—The well-known butterfly and cylindrical valves are yet in general use, the cylindrical type having been adopted for the great Panama Canal locks. In 1883, Mr. F. G. M. Stoney, of London, England, invented the sliding valve, known as the "Stoney gate". This type is also widely used, being at times fitted with roller bearings.

The mechanisms for opening and closing lock-gates have been greatly improved, the bull-wheel type (devised by Edward Schildhauer, M. Am. Soc. C. E., for the Panama Canal locks*) operating either chains or spars, being greatly favored for operating heavy gates.

Almost all important locks now have some form of a removable, emergency dam for closing the lock at its upper pool end in case of accident to the gates. The Panama locks are also provided with guard chains to prevent the accidental ramming of the gates.

In addition to these precautionary measures, ships passing into and through the Gatun Locks of the Panama Canal are not permitted to use their own power, but are towed by locomotives operating on tracks laid on the lock-walls.

It has also become customary in the larger locks to operate the gates and valves by power generated from the head between the upper and lower pools.

Fixed Dams.—The major advance in construction methods for fixed dams has been in the development of dams of the single and multiple-arch types. The construction of earth dams by hydraulic pumping methods should also be noted. The Gatun Dam of the Panama Canal is a notable example.

Sea Walls and Breakwaters.—About 1890, the late Brig.-Gen. W. L. Marshall, U. S. A. (then Captain Corps of Engineers, U. S. A.) devised a shore protection for the water-front of Chicago, Ill., which marked an advance in work of that character. It had been customary to oppose wave force by the size and weight of a sea-wall, requiring expensive construction. Captain Marshall tripped the incoming wave by means of a stone-filled crib with its top at the level of the lake surface. The energy of the wave was then exerted downward and was taken up by the paving on top of the crib and a stone paving to the rear laid on a gentle slope. At the crest of the slope only a light wall was needed to stop the final wash of the wave. The same principle was followed by the writer in constructing the Malecon of Havana, the sea-wall being set back about 35 ft. from the front of the slightly inclined rock surface lying at about high-water level, on which the wall was built.

In some sea-walls wave action is partly deflected by building the vertical wall with a concave face. In the breakwaters of the harbors of the Great Lakes, the old vertical crib walls have been abandoned and the breakwaters are made lower, with an inclined exposed face.

Economy has been effected and durability increased in these breakwaters by substituting for the old form of timber cribs, caissons made of reinforced concrete, which after having been placed are filled with concrete in mass.

Reinforced concrete caissons are also being used as subaqueous foundations for walls and for piers. Examples of this use are found in the dock wall at

* Paper No. 20, International Eng. Congress, San Francisco, Calif., 1915; Report, Isthmian Canal Comm., 1910, p. 53.

Petosky, Mich.*, and in the sea-wall for the China Basin, San Francisco, Calif.†

Messrs. Harrison S. Taft and O. D. Jones have written a valuable review of the use of concrete caissons,‡ and Mr. Taft's paper on "Reinforced Concrete Docks",§ presented to the Society, contains much historical data.

The progress of waterways engineering in the United States is reflected in the work accomplished. The work done has been widely spread over the entire country, as must be the case, since the care and improvement of waterways, under the Constitution, is the duty of the Nation. Appropriations for this class of work are made from the National Treasury, and the will of the people at large is that no part of the widespread territory shall be without receipt of some direct benefit from the expenditures for waterways improvements. Under this condition it has been impossible to finish individual projects for improvement as rapidly as would be beneficial were it possible to consider the needs of each improvement separately and allot funds for it irrespective of other demands. Nevertheless, the accomplishments have been great and have materially assisted the economic welfare of the people, as shown by the recorded increases of trade reported in the commercial statistics for each port and waterway published annually by the Government.‖

Space will not permit an enumeration of all work done. A few examples must suffice. Almost all the work mentioned has been done since 1865.

Almost all the harbors of the Great Lakes are at the mouths of rivers. These were obstructed by bars, largely formed by the drift of sand along the coasts under storm action (drift and wave bars). The channels across the bars were shifting, with depths of from 3 to 5 ft. The earliest improvements were to fix and deepen these channels by jetties and to deepen the rivers above the mouths by dredging. Later, a system of breakwaters was built outside the mouths, forming exterior harbors. All of the more important harbors now have a navigable depth of 20 or 21 ft. below the standard lake level. The channels connecting the Upper Lakes have been improved to the same navigable depth. For many years, the navigation between Lakes Erie and Ontario has been through the Welland Canal, lying in Canada. The Canadian Government is now engaged in enlarging the channel and locks so as to permit navigation by ships of 25-ft. draft. The total commerce of the United States ports on the Great Lakes in 1924 amounted to 203 534 234 tons,¶ valued at $3 572 521 000, in addition to a ferry traffic of 926 140 tons, valued at $125 797 000.

The general program for the rivers of the Atlantic Coast is to improve each to the extent justified by present and prospective commercial use and to connect the mouths of all between Boston, Mass., and the Rio Grande, in Texas, by an intra-coastal barge canal. The Mississippi River and its principal tributaries are being improved so as to form a system of barge naviga-

* *Engineering News-Record,* Vol. 88, 1922, p. 956.

† *Loc. cit.,* Vol. 91, 1923, p. 252.

‡ *Professional Memoirs,* Corps of Engrs., U. S. A., Vol. VII, 1915, p. 145.

§ *Transactions,* Am. Soc. C. E., Vol. LXXVIII (1915), p. 1058.

‖ Rept., U. S. Chf. of Engrs., 1925, Pt. 2.

¶ *Loc. cit.,* pp. 15 and 17.

tion from Pittsburgh, Pa., Chicago, St. Paul, Minn., and Kansas City, Mo., to the Gulf of Mexico, with such improvement of minor tributaries as may be economically justifiable. The rivers of the Pacific Coast are treated individually, no great system of river grouping being possible.

Table 4 shows some of the accomplished work. The depths given are for low water.

TABLE 4.

River.	Part improved.	Natural controlling depth, in feet.	Present controlling depth, in feet.
Hudson	Hudson to Troy, N. Y.	3.5	11.5
Delaware	Philadelphia, Pa., to sea, 101 miles	17	32.5
Delaware	Trenton, N. J., to Philadelphia, 305 miles.	3	10.5
Savannah (Ga)	Savannah to ocean	10	25.6
St. John's (Fla.)	Jacksonville to ocean	5 to 7	30
Mississippi	New Orleans, La., to Gulf:		
	South Pass	9	33
	Southwest Pass	9	38
	Cairo, Ill., to New Orleans	4.5	9
	Mouth of Missouri to Cairo	3.5	8
	Minneapolis, Minn., to mouth of Missouri	2.5	5
Ohio	Pittsburgh, Pa., to Leavenworth, Ind.	1 to 2	9
Alleghany		3	4 to 7
Monongahela		12	6 to 8
Great Kanawha		3	6
Kentucky	For 280 miles above mouth	Unnavigable.	6
Cumberland	Below Nashville, Tenn	0.5	4
Tennessee		1	4
Missouri	Below Kansas City, Mo	2.5	3.5
Illinois		1.66	8
Sacramento (Calif.)		4	7
Columbia (Ore.)	To Portland on Willamette River	15	28

The construction of the Panama and Cape Cod Ship Canals and the enlargement of the New York State Barge Canal are all recent works. Some of the improvements made in ocean ports are given in Table 5.

TABLE 5.

Port.	Natural controlling depth of entrance channel, in feet.	Present controlling depth, in feet
Boston, Mass	23	35
New York, N. Y	23.6	40
Baltimore, Md	17	34
Norfolk, Va., (Entrance channel)	30	38
Norfolk, Va., (Inner channel)	21	40
Charleston, S. C	12	30
Mobile, Miss	5.5	29
Galveston, Tex	9	30
Los Angeles Calif., (Inner harbor)	2	30
San Francisco, Calif., (Main channel)	32	40

ACKNOWLEDGMENTS

The writer is indebted to Capt. W. S. Crosley, U. S. N., Hydrographer, Navy Department, to Maj. Milo P. Fox, Corps of Engineers, U. S. A., and to R. L. Faris, M. Am. Soc. C. E., Acting Director, U. S. Coast and Geodetic Survey, for valuable assistance in the preparation of this paper.

T

Th
York.
an es
Henle
prome
acteri
is bro
it un
rapid
strean
delph
is a
the S
enter
Th
is ap
miles
A
throu
the t
havin
enter
at th
T
tidal

N
*
1926.
t

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE DELAWARE RIVER FROM PHILADELPHIA TO THE SEA*

By F. C. Boggs,† M. Am. Soc. C. E.

The Delaware River has its origin in a small lake in Southeastern New York. It flows in a general southerly direction, emptying into Delaware Bay, an estuary of the Atlantic Ocean, the entrance to which is marked by Cape Henlopen on the west and Cape May, the southern end of the New Jersey promontory, on the east. The upper reaches of the river have the usual characteristics of a mountain stream. Its middle section is less torrential, but is broken by reefs and rock ridges which, with its shallow depths, render it unfit for navigation except for small boats in the quiet pools between the rapids. At Trenton, N. J., the river begins its long meander as a tidal stream through the low coastal plain bordering the ocean. From Philadelphia, Pa., to the sea, the section with which this paper is concerned, it is a stream of varying depths increasing in width from 5 000 ft. just below the Schuylkill River near Philadelphia, to approximately 4 miles where it enters the Bay.

The total length of the waterway from the Atlantic Ocean to its source is approximately 365 miles, of which the stretch comprising the first 131 miles from the sea to Trenton, is tidal.

Although there are numerous streams flowing into the Delaware River throughout its entire length, there are two only which can be dignified by the term tributary rivers. These are the Lehigh and the Schuylkill, each having its source in the Blue Ridge Mountains in Pennsylvania, the former entering the main stream at Easton, 50 miles above Trenton, and the latter at the southern boundary of the City of Philadelphia.

The discharge of the Delaware River at its mouth is a combination of tidal and fresh-water flow. No data are available on which to base an exact

Note.—Written discussion on this paper will be closed in February, 1928.

* Presented at the meeting of the Waterways Division, Philadelphia, Pa., October 8, 1926.

† Col., Corps of Engrs., U. S. A.; Dist. Engr., Philadelphia, Pa.

determination of the fresh-water flow, but that it is a material factor is evidenced by tidal and current records at Liston Point, the section where the waterway designation is changed from river to bay. At this point the duration of ebb is 6.6 hours as against 5.8 hours for flood, while the ebb current at strength in a vertical section in the channel averages approximately 2.4 knots as compared with the flood of only 1.8. The combined discharge at this section during hours of ebb is at the rate of about 800 000 cu. ft. per sec., and a very rough approximation gives the fresh-water discharge during the same period as averaging nearly 250 000 cu. ft. per sec.

Original Condition

In its original condition, the Delaware River below Philadelphia was obstructed by a number of shoals or bars, over which the minimum usable depths at mean low water were between 17 and 22 ft. Fifty years ago, before the systematic improvement of the river was undertaken by the Federal Government, a ship sailing up the channel to Philadelphia required in some cases four days to make the trip, owing to the necessity of waiting for high water to pass over these shoals.

During the winter months the ship was also likely to be delayed a day or more by ice jams blocking the entire channel at the "Horse Shoe", where the river bends sharply to the westward at the lower end of Philadelphia Harbor. The harbor itself was obstructed by Smith and Windmill Islands and their adjacent longitudinal bars, and by shoals in its upper reaches.

River Bottom

Except at a few localities where there are rock outcrops, the bed of the Delaware River below Philadelphia is composed of sand and gravel overlaid in most sections by a light mud. At some places this overlying mud extends to considerable depths, rendering the construction of dikes and bulkheads difficult and costly. This is particularly true where these works are required to sustain the thrust of earth fills behind them.

Omitting the rock sections, dredging can readily be done by the suction type of dredge, although for contract work clam-shell and dipper dredges have been used almost exclusively for the removal of material from the channel.

At times muddy, the Delaware River below Philadelphia can not be classed as a silt-bearing stream, although in time of freshets in the upper river, or in tributary streams, some deposit is undoubtedly due to silt brought into the river by flood waters. Caving banks and the washing of the banks by waves are also unquestionably the sources of additional deposits.

The shoaling in the channel which must be removed to maintain the project depth is caused partly by these deposits, but the large bulk of such shoaling cannot be accounted for from these sources alone. The tendency of cross-currents to wash the material from the sides of the bed into the channel has been noted by many investigators. The writer believes that the largest part of the dredging for channel maintenance is due to two factors: First, the building up by these cross-currents, and by deposition of sus-

pended material, of a middle ground where, due to islands or to the peculiar bends in the river, the ebb flow and the flood flow do not naturally take the same channel; and, second, the gradual movement of the surface of the mud bottom toward the dredged channel, resulting in a shoaling of the channel and a general lowering of the bed of the stream to some distance on each side of the channel width. This latter condition is shown by a comparison of surveys made in 1909 and 1924.

Tides and Currents

A complete and exhaustive study of tides and currents in the Delaware River has recently been made by the United States Coast and Geodetic Survey.* A copy of this report was loaned to the writer by the courtesy of E. Lester Jones, M. Am. Soc. C. E., Director of the Survey, and what follows under this heading is based almost entirely on this report.

The oldest recorded tidal observations in the Delaware River Basin date back to about 1835. These early records are based on unimproved instruments and methods and are probably far from accurate. The introduction of the automatic, self-registering tide gauge in 1854 and the adoption of standard time about 1885 permitted a more nearly accurate record.

The tide from the open ocean enters Delaware Bay between Cape May on the New Jersey shore and Cape Henlopen in the State of Delaware, a width of about 10 nautical miles. The sweep of the tide follows the estuary of Delaware Bay which widens gradually from the Capes to a maximum width of 23 miles, this width decreasing to about 4 miles at the upper end of the Bay where it merges with no definite topographic feature into the Delaware River. Following the river the tidal wave moves over a generally, but in no case an abruptly, decreasing front until at Philadelphia the width of the river is about 2 000 ft., and at Trenton, the head of the tidal flow, is about 1 000 ft.

A study of the tides at Philadelphia was facilitated by the fact that there is available an almost continuous record from 1901 to 1920. These records are of particular value because they cover much of the period of channel improvement by dredging and the introduction of training works, and afford some opportunity to study the tidal regimen as affected thereby.

The mean lunitidal interval, that is, the interval of time between the transit of the moon and the following high or low water, is given for Philadelphia for high water as 1.5 hours for the first ten years and 1.47 for the second. For low water the corresponding figures are 9.01 and 8.94 hours, respectively. The periodic seasonal change is readily grasped from a study of the graph in Fig. 1.

The authors of the study believe the variations in the lunitidal interval from month to month are due to the variation in the frictional resistance to the tidal wave caused by differences in river level, and they point out that in the summer months when the river levels are highest, the intervals are the least, while the reverse is true for the winter months.

* "Tides and Currents in Delaware River and Bay," by L. M. Zeskind and E. A. Le Lacheur, U. S. Coast and Geodetic Survey *Special Publication No. 123*, 1926.

The duration of the rise and the fall of the tide at Philadelphia is given in Table 1.

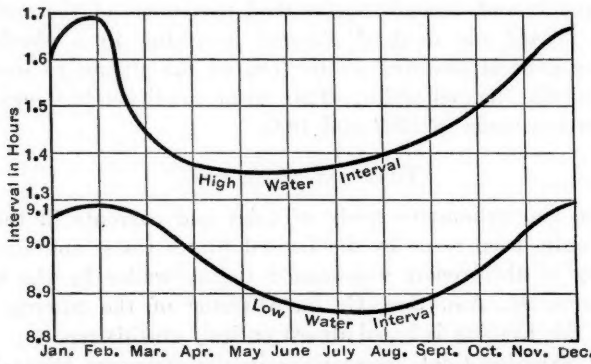


FIG. 1.—ANNUAL VARIATION IN LUNITIDAL INTERVALS, PHILADELPHIA, PA.

Table 1 is typical of river tides where the constant fresh-water flow opposes the flood and runs with the ebb, thus shortening the time of the former and increasing that of the latter. It is interesting to note that while for each full month the sum of the time of rise and fall equals 12.42 hours, a tidal cycle, yet daily records of individual tidal cycles show in some cases considerable variations, the time of an individual rise or fall being, in a few instances, one or two hours greater or less than the monthly mean, due to the effect of the wind advancing or retarding the tidal movement of the water.

TABLE 1.—DURATION OF RISE AND FALL OF TIDE, PHILADELPHIA, PA., ANNUAL VARIATION.

Month.	Duration of rise, in hours.	Duration of fall in hours.	Month.	Duration of rise, in hours.	Duration of fall, in hours.
January	4.96	7.46	July	4.92	7.50
February	5.01	7.41	August	4.93	7.49
March	4.79	7.63	September	4.98	7.44
April	4.81	7.61	October	4.99	7.43
May	4.89	7.53	November	5.00	7.42
June	4.91	7.51	December	4.98	7.44

To the navigator, the characteristics of the tidal movement in which he is most interested are the low-water and the high-water levels. The annual variations in the plane of low water for Philadelphia for the period, 1901 to 1920, are shown in Fig. 2 (*a*), and corresponding variations in high water are shown in Fig. 2 (*b*). A study of these diagrams is interesting. It is seen that the annual variation in high-water level is more than twice that for the low-water level, and that while the former appears to run through the year in a single cycle, the latter has two clearly marked cycles. Both are a minimum in February and while the maximum high is in June, yet the April high is very close to the June figure and corresponds with the

maximum low which occurs in April. Here, however, the similarity of the curves ceases and while the high remains at nearly a constant maximum from April to September, the curve of low water approaches the minimum in July. This difference is undoubtedly due to the fresh-water flow of the river.

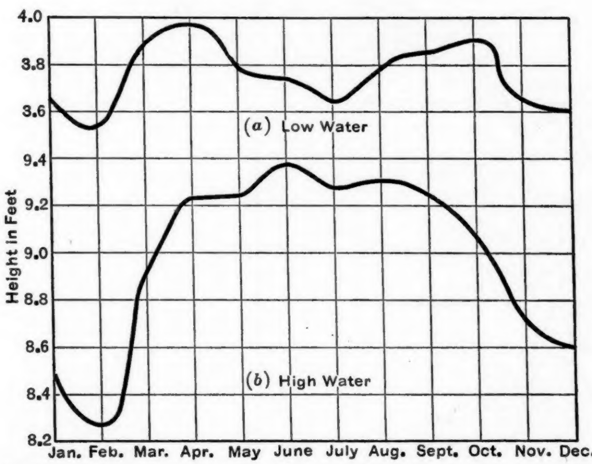


FIG. 2.—ANNUAL VARIATIONS IN LOW, AND IN HIGH WATER, AT PHILADELPHIA, PA.

The highest water recorded on the Philadelphia gauge was on October 11, 1903, when the river rose 4.22 ft. above mean high, or 9.45 ft. above the mean low for the period, 1901 to 1920. The lowest record for the same period was on March 29, 1919, on which date the water fell 4.05 ft. below the mean low-water plane for that period. The mean range of tide for any year of the period may be obtained by subtracting the mean low for that year from the mean high for the same year. This range varies between 5.03 and 5.34 ft., averaging 5.23 ft. for the 20 years.

From a brief glance at the map of the Delaware River (Fig. 3), it is evident that northwest winds tend to drive the water out of the river, thus producing a tide lower than usual, while an east wind tends to drive the waters up the Bay and river, thus causing a high water higher than usual. Additional tables and discussions in the report from which the data contained herein were extracted, show these phenomena very clearly. An interesting case is cited in that report. During the blizzard in Philadelphia in March, 1914, the wind began blowing from the northwest on March 1 at the rate of 25 miles per hour and at 6:00 A. M., on March 2, it reached a velocity of 40 miles, which was maintained for a considerable time. As a result there was only one high water and one low water between 4:00 P. M., March 1, and 2:00 P. M., March 2, instead of the usual two high and two low, the water falling steadily during the entire 22 hours, dropping almost 4 ft. below normal.

The foregoing discussion has been confined to results obtained on records from the Philadelphia gauge which are continuous from 1901 to 1920. Records

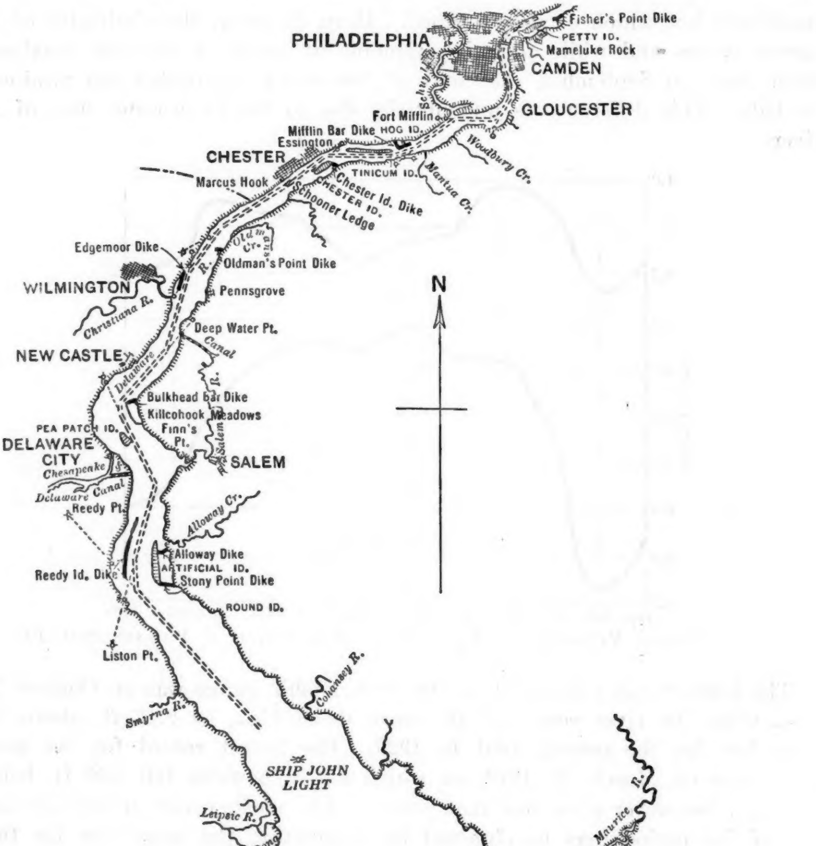


FIG. 3.—MAP OF DELAWARE RIVER AND BAY.

of the river below Philadelphia are not so complete, but even under these conditions some of the deductions are very interesting.

TABLE 2.—TIDAL DATA, DELAWARE RIVER AND BAY.

Location.	Nautical miles below Philadelphia.	Lunitidal high water interval, in hours.	Low water interval, in hours.	Duration of rise, in hours.	Mean range, in feet.	Observation.	
						Date.	Length.
Delaware River:							
Fort Mifflin, Pa.	8	....	....	4.92	5.93	1879-80	14 months
" " "	..	0.83	8.32	4.93	5.88	1882-83	6 "
" " "	..	0.96	8.37	5.01	*5.17	1924	3 "
Marcus Hook, Pa.	18	12.25	7.29	4.96	6.06	1878-79	1 year
" " "	..	12.03	7.36	4.67	6.08	1881	10 days
" " "	..	0.09	7.42	5.09	5.51	1915-16	1 year
" " "	..	0.08	7.44	5.06	†5.41	1924	3 months
New Castle, Del.	29	11.57	6.18	5.39	5.50	1840	5 "
" " "	..	11.56	6.39	5.17	6.00	1873-86	2 weeks
" " "	..	11.64	6.52	5.12	†5.42	1924	3 months
Reedy Island Quarantine, Del.	39	10.65	5.27	5.38	5.91	1896-1900	4 years
Reedy Island Quarantine, Del.	..	10.94	5.57	5.37	*5.56	1924	3 months
Delaware Bay:							
Mahon River Lighthouse, (Western Shore)	61	9.94	3.50	6.44	5.64	1841-43	5 months
	..	9.39	3.50	5.89	5.31	1883	2 "
Fortescue Beach, (Eastern Shore)	62	9.06	3.17	5.89	6.00	1880-82	1 "
Mispillion Light, (Western Shore)	74	8.72	3.85	4.87	3.84	1883	17 days
Maurice River Light, (Eastern Shore)	67	8.80	2.84	5.96	5.60	1884-85	2 months
Delaware Breakwater, (Western Shore)	86	7.88	1.63	6.25	4.16	1863-68	1 "
Cape May Steamboat Landing, (Eastern Shore)	82	7.83	1.97	5.86	4.65	1836	1 "
	..	7.91	1.74	6.17	4.74	1867	10 days
	..	8.29	1.91	6.38	4.78	1883	3 "
	..	8.16	1.90	6.26	4.71	1885	1 month

* Ranges based on 3 years of high and low water.

† Ranges based on 8½ years of high and low water.

From a study of Table 2 it is noted that both the time and range of tide increase from the Capes. It is also noted that for corresponding stations on the two shores of Delaware Bay, those on the eastern shore have a greater range of tide than those on the western shore. This is attributed by the authors of the study to the effect of the rotation of the earth, which tends to deflect all moving bodies to the right in the northern hemisphere, thus piling the flood tide on the eastern shore and increasing high water there, and likewise deflecting the ebb tide to the western shore and causing a higher low water on that shore. Above the Bay the waterway is considerably narrower, and this condition is not so apparent.

Another peculiarity noticeable from Table 2 is the decrease in range of tide for observations made after 1900 as compared with those made prior to that date, for stations in the Delaware River. Corresponding to this variation in range, there is also observed a change in tidal datum planes. This is made clear in Table 3.

It is noted from Table 3 that, while the plane of mean high water has been little affected, the plane of mean low water is materially higher in all

cases in the series following the year 1900 over the plane for the series preceding that year. This is further verified by the experience of the Army Engineers in charge of the improvement of the Delaware River. Their records corroborate the observations of the U. S. Coast and Geodetic Survey, at Philadelphia, namely, that the tidal range has decreased, and the elevation of mean low water has been raised from 0.5 ft. at some points to 0.8 ft. at others, as compared with the observations of 1880 and earlier. The old datum, however, generally referred to as "mean low water at Marcus Hook—1879", has continued in use and still forms the plane of reference for depths appearing on Government charts. As there would be little, if any, advantage and some disadvantages in changing this datum plane, it has been retained. The result is not detrimental to navigators, since there is at actual mean low water a slightly greater depth than shown on the charts.*

TABLE 3.—Changes in Tidal Datum Planes.

Station.	Referred to.	Series.	Mean tide level, in feet.	Mean high water, in feet.	Mean low water, in feet.
Philadelphia, Pa.......	Staff of 1901...........	1891-92	6.07	8.91	3.23
		1901-20	6.36	8.96	3.76
Marcus Hook, Pa......	Staff of 1878...........	1878-79	3.12	6.10	0.14
		1915-16	3.42	6.17	0.67
New Castle, Del........	Bench Mark No. 4.....	1840-86	13.06	16.04	10.08
		1916-24	13.70	16.42	10.99

Current velocities in the section of the Delaware River between Philadelphia and the sea vary in the channel for flood at time of strength from 1½ knots in Lower Delaware Bay to 2 knots in the river proper. Channel currents at ebb strength are usually somewhat greater than at flood. In general, the vertical curve of velocities in the channel for the ebb shows a rather rapid decrease in strength as the bottom is approached, whereas for flood conditions this is much less marked and at some points near the mouth there is even a decided increase in strength at considerable depths over the surface strength. This is as would be anticipated in a river emptying into the sea and having a fair percentage of fresh-water discharge.

River Studies

Before discussing the actual work of improvements on the Delaware River, it may be interesting to refer briefly to certain theoretical investigations which have been made from time to time.

About 1878, the late Henry Mitchell, M. Am. Soc. C. E., Assistant in the U. S. Coast and Geodetic Survey, made a study of the Delaware River from Fort Mifflin, 8 miles below Philadelphia, to the mouth. He followed this at a somewhat later date by other studies based on new and more complete hydrographic surveys which had not been available at the time of his

* See "Tides and Their Engineering Aspects," by G. T. Rude, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1105 and Fig. 21.

original report.* As a result of his first study, Mr. Mitchell laid down the following rules for this section of the river:

First.—The transverse section is directly proportional to the discharge.
Second.—The width is proportional to the discharge.
Third.—The mean depth is the same in all sections.

Mr. Mitchell pointed out as the most interesting feature of these studies the "constancy of mean depth". It is not intended by this statement, nor by Mr. Mitchell's third rule, to imply that mean depths were actually the same in all sections, as at various points there were some variations due chiefly to inequalities of the soil. However, by plotting mean depths along the length of the river, he shows that a horizontal straight line at a depth of 18.64 ft. would be the correct rectification of the irregular curve of actual mean depths.

Starting with this assumption of constancy of mean depth, the width of the river becomes the independent variable. Actual widths at various sections were plotted, and a rectified curve of widths using the plotted points as guides was drawn. Mr. Mitchell found that this curve corresponded to a parabolic formula. The theoretical cross-sections are obtained by multiplying theoretical widths by the mean depth of 18.64 ft.

The practical application of these studies is stated by Mr. Mitchell as follows:

"The persistent tendency to constancy of mean depth throughout the whole length of the estuary would seem to discourage the hope of improvement by dredging. * * * Except for two or three shallow reaches of short extent, there is plenty of water from the ocean to Philadelphia, and at these obstructed reaches, artificial contraction of the water way may be made without altering the course of the stream or sensibly reducing the tidal volume; so that deepening may be induced where required without changing the conditions elsewhere".

Further studies were made in 1882 by Mr. E. A. Giesler, Assistant Engineer, under the Corps of Engineers, U. S. A. He called attention to the fact that, while there was a definite increase in cross-sectional areas going toward the mouth, yet the increase was not a steady one, but was subject to frequent and irregular fluctuations which he very pertinently stated might be due to local causes. He felt that these local fluctuations might confuse the result, and, therefore, in order to eliminate them, he used instead of the actual cross-section at any given point, the mean of ten cross-sectional areas, five above and five below the section under consideration. He also based his theoretical smooth curve of increasing cross-sections on three sections at widely separated intervals of distance on account of their position on the more uniform parts of the curve developed by using mean cross-sections obtained as just described. From his study Mr. Giesler determined that the curve showing the increase in cross-sectional area of the river from Philadelphia to the mouth approached very closely an hyperbola.

* U. S. Coast and Geodetic Survey Report, Appendix No. 8, 1883, and Appendix No. 12, 1885.

Frank C. Warner, M. Am. Soc. C. E., Assistant Engineer in the Engineer Department of the Army, in 1917, modified Mr. Giesler's figures by using the mean of three cross-sections above and three below the section considered. He adopted an hyperbolic formula for increases in sectional areas similar to that used by Mr. Giesler. Mr. Warner states:

"The practical application of the theoretical law thus obtained must be made only in connection with studies of local conditions, or otherwise the results may be erroneous. Applied to a straight tidal river channel with uniform conditions of bed and banks throughout its whole length, definite and reliable results may be expected, but when applied to the Delaware River channel, in which variable conditions exist, such as sharp bends and bed and banks composed of materials varying from soft mud to sand, gravel, boulders, and even ledge rock, it is apparent that such theoretical law can be applied only as a check, but reliable in so far as it will determine limits in regulation beyond which it will not be safe to go with due regard to the regimen of the river as a whole".

H. F. Flynn, M. Am. Soc. C. E., who was Assistant Engineer on the work on the Delaware River, in 1918 called attention to the fact that Mr. Warner's results were based on a survey made in 1916, after a considerable amount of work had been done, including dredging and dike construction. He believed that the natural regimen of the river should be used instead of an artificial state. He noted also that investigators following Mr. Mitchell had used three sections at widely separated points on the river for determining their curves of areas, and he objected to this method for the reason that such sections do not "necessarily indicate that the river has a proper cross-section at those points, but rather that the relation of width to area happens to be such that the river is able or rather forced to maintain them".

In his study, Mr. Flynn goes back to the cross-sectional areas and widths used by Mr. Mitchell based on the survey made about 1880 before the natural state of the river had been materially affected by artificial works. He then, by the method of least squares, developed the following formula for the theoretical curve of increasing cross-sections:

$$X = \frac{a - b\,y}{y + c}$$

in which,

a, b, and c are constants;

X represents the cross-section of the river at any point, in hundreds of square feet; and,

Y represents the distance along the center line of the channel from the place of origin, in nautical miles.

Table 4 gives the results obtained from the formulas developed by these investigators and Fig. 4 shows the relation between the theoretical curves of increasing cross-sections resulting therefrom.

For cross-sections Mr. Flynn adopted the statement by Mitchell that in straight reaches a sine curve coincides very closely with the natural bottom. From this hypothesis the following formula was derived,

$$W = \frac{\pi\,A}{2\,D}$$

in which,

D = maximum depth, in feet.
A = area of cross-section, in square feet.
W = surface width, in feet.

TABLE 4.—THEORETICAL AREAS OF CROSS-SECTION AT HALF TIDE.

Distance below Philadelphia, in nautical miles.	Square Feet.		
	Mitchell.	Warner.	Flynn.
5	100 000	104 800	106 800
10	114 000	119 900	124 200
15	137 000	138 300	145 200
20	170 000	161 100	171 200
25	213 000	190 100	204 000
30	264 000	228 400	246 700
35	325 000	281 100	304 700
40	396 000	358 300	387 900
45	476 000	....	517 400
50	565 000	....	746 500

Mr. Flynn adds:

"It is recognized that there are many elements affecting the width of the river that have been neglected in the above method of location of training dikes. The effect of curvature has not been taken up, but it is not large. The matter of the location of the deepest part of the channel has been disregarded. It is believed, however, that the method may be used to advantage. It is better adapted to the upper reaches than to the broader portions below. In the narrow, upper portions of the river the dredged portion of the channel forms a considerable fraction of the total width between banks. Here the contraction required to make the channel self-maintaining, that is, to make the distance between the banks such that it will correspond to a natural depth of 38 ft. at half tide, is comparatively small. Proceeding down stream, however, conditions change, until at the lower end we find that the width of the dredged channel is a very small fraction of the total width, and the dredged channel assumes more the form of a ditch or canal dug through the comparatively flat river bed. Any attempt to create a natural depth of 38 ft. at half tide would require so considerable a reduction in width between banks as to be impracticable."

In an interesting paper on the "Relation of Depth to Curvature in Channels", H. C. Ripley, M. Am. Soc. C. E., states, under the heading, "Conclusions":*

"Professor Mitchell has shown,† in his investigations of the estuary of the Delaware River embracing a length of 52 miles from Philadelphia, Pa., down 'that the mean depth is the same in all sections and is equal to 18.64 ft. from all soundings for 46 nautical miles * * *'.

"Now, as the maximum depth in the straight reaches of any stream is equal to the mean depth multiplied by a constant, and the maximum depth in bends is equal to the mean depth multiplied by the same constant plus the effect due to curvature, it follows that the maximum depth in all straight reaches will be the same. Any attempt, therefore, to deepen the channel

* *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 238.

† U. S. Coast and Geodetic Survey Report, 1883, pp. 239–245.

by dredging will disturb this persistent tendency to uniformity of depth. Hence, it would appear that it is not practicable to secure any permanent improvement in the navigable channel of that river by dredging. It is evident, therefore, that the logical method would be to convert the straight reaches into channels of suitable curvature by means of training walls and to let Nature do the rest. In this way equality of mean depth would be preserved and the increased depth secured would remain permanent forever."

Further reference to these theoretical studies will be made subsequently.

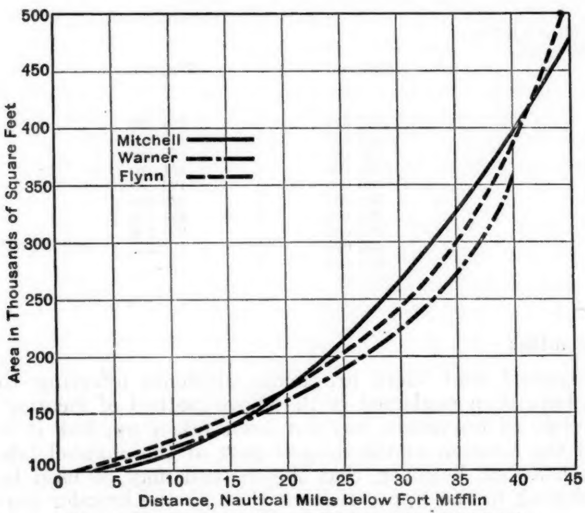


FIG. 4.—CURVES OF THEORETICAL CROSS-SECTION.

PROJECTS

The first project for the systematic improvement of the Delaware River was adopted in 1885. Previous to that year, and commencing as early as 1836, Congress had appropriated funds and work had been done on isolated improvements, such as ice piers and the dredging of channels across the more obstructive shoals.

The River and Harbor Act of July 5, 1885, however, carried an appropriation for the "improvement of Delaware River from Trenton, N. J., to its mouth", and a project was accordingly prepared by a Board of Engineers appointed to consider the subject. This project provided for the construction of a channel having a least depth of 26 ft. at mean low water, and a least width of 600 ft. to extend from the Port Richmond grain piers, in Philadelphia Harbor, to deep water in the Bay, a distance of about 60 miles. This project was prepared a short time after the studies made by Mr. Mitchell and Mr. Giesler, and it is interesting to note the influence that these studies may have had on the proposed work. The late Col. William H. Heuer, U. S. A. (*Retired*), M. Am. Soc. C. E. (then Major, Corps of Engineers, U. S. A.), in his preliminary examination refers to Mr. Mitchell's study, and Mr. Giesler's investigations form a part of Major Heuer's report. Major Heuer states:

"The remedy for improving the channelway is contraction, so as to compel both inflowing and outflowing tides to occupy the same channel".

While the report of the Board of Engineers which reviewed Major Heuer's report, and on which the adopted project was based, made no direct reference to the studies by Mitchell and Giesler, they were undoubtedly in mind and possibly used in the location of some of the dikes recommended.

The improvement was in general to be secured by the construction of low-water dikes in the vicinity of permanent shoals, aided by dredging if the training works alone proved incapable of producing results, supplemented by rock removal at one shoal. The project as submitted by the Board involved the construction of five longitudinal dikes and possibly one in addition. As having some bearing on the studies of the theoretical widths of the river, the Board made the following statement in regard to the shoal at Cherry Island Flats, a few miles above Wilmington, Del., through which a self-sustaining channel had previously been dredged:

"This permanence may be considered remarkable where the river is so wide and straight, and is mainly due to the judicious selection of the location of the dredged channel and the agreement in direction of flood and ebb currents."

The theoretical studies indicate the necessity of contraction at this place, but experience with the 26-ft. channel and, later, with the 35-ft. channel, have shown that these channels are practically self-sustaining through this shoal.

The history of the 26-ft. channel work is chiefly interesting as demonstrating the ineffectiveness of low-water dikes, and the fact that even after raising these works to high water, dredging was necessary to produce the desired depth in any reasonable time. In justice to the original conception it should be added that once the depth was obtained by dredging, the dikes assisted in maintaining it, and it may be that the failure of the contracting works in the first instance was due to character of material.

Before the completion of the 26-ft. project the growth of Philadelphia as a port, and the general change from sailing vessels to steam with increased draft, had awakened a demand for a deeper channel. In 1899, Congress authorized a new project superseding the one of 1885, this new project providing for a channel 30 ft. deep and 600 ft. wide from Philadelphia to deep water in the Bay, at an estimated cost of $5 810 000.

When the 30-ft. project was adopted, about $1 600 000 had been expended on the 26-ft. channel, which depth had been secured over widths varying from 200 to 600 ft. from Philadelphia to the Bay, except for uncompleted lengths aggregating about 4 miles. In addition, the harbor at Philadelphia had been improved by the removal of Smith and Windmill Islands, by the establishment of pierhead and bulkhead lines, and by the partial removal of Mameluke Rock (Fig. 3), an area which interfered with the movement of vessels in the upper harbor. The report on which the 30-ft. project was based was submitted by a Board of Engineers, which had before it a preliminary study

made by the late C. W. Raymond, Brig.-Gen., U. S. A. (*Retired*), M. Am. Soc. C. E. (then Lieutenant Colonel, Corps of Engineers, U. S. A.). In this study Colonel Raymond stated:

"The greatest depth which could probably be maintained at reasonable cost might perhaps be determined by a study of the river itself, if the form and dimensions of the channel were solely the result of the action of flowing water, but this is not the case. Natural obstructions, immovable by water, such as the rock at Schooner Ledge, exist in some parts of the channel; and in other places where bars have persistently formed, old wrecks and other artificial obstructions have been found by dredging. Such artificial obstructions have been removed at Five Mile Bar, Smith Island Bar, Bulkhead Bar, and Reedy Island Bar, and doubtless other obstructions of a similar character will be discovered as the channel improvement progresses. For this reason the various attempts heretofore made to determine the theoretical form and dimensions of an improved channel in the Delaware River from a study of existing cross-sections have been found after careful examination to be of little practical value."

He also states:

"In connection with this question of channel maintenance, it may be asked whether the depths could not be maintained by the construction of dikes properly located to direct the currents and confine their action within the limits of the channel. The original project of the Board of Engineers of 1885, in general accordance with which the improvement of the Delaware River has been thus far conducted, provided for a very large amount of dike construction. Dikes have accordingly been built at Five-Mile Bar, Mifflin Bar, and Bulkhead Bar, and they have admirably fulfilled the purposes for which they were designed. It will be observed, however, that these dikes are all located in the part of the river above the limit of effective inflow of the waters of the marine tide wave. In the lower part of the river the conditions for dike construction are much less favorable. The dikes would have to be of great length. They would be difficult and very expensive to construct, on account of the softness of the bottom, and they would have to be located with the greatest skill and care to avoid obstructing the inflow of the flood tide. Owing to the action of ice and storms these structures would require large expenditures for annual repairs. Moreover, it has been found practically impossible to locate and construct such works on the lower part of the river on account of the real or fancied grievances of riparian owners. Accordingly the Board of Engineers of 1896 recommended the abandonment of dike construction in this part of the river and the improvement of the channel by dredging. When the Board of Engineers of 1885 submitted its report, the cost of dredging was relatively so great as to amply justify the expense of dike construction. With the improved dredging machinery and low prices of the present time the construction of dikes is no longer generally considered advisable for the improvement of a bar within the mouth of a tidal river."

The Board recommended as a part of the project a longitudinal bulkhead near the Pennsylvania shore, just above the Christiana River, and also one near the New Jersey shore nearly opposite Reedy Island (Fig. 3). These bulkheads were to serve two purposes: First to act as training dikes; and, second, to form basins into which material dredged from the channel could be deposited. The latter dike finally took the form of an enclosure in the river which has since been raised by those deposits into an island very properly known as Artificial Island (Fig. 3). The bulkhead above the Christiana River was started, but was not completed. The chief matter of interest during the work

on the 30-ft. project was the introduction of the sea-going hopper dredge. This will be described later.

The demands for a channel to accommodate the deeper draft vessels required by the economics of ocean-borne commerce, again became insistent, and an investigation was made with a view to deepening the channel to Philadelphia to 35 ft. The result was the existing project which was adopted in 1910. It provides for the formation of a channel 35 ft. deep from Philadelphia to deep water in Delaware Bay, a distance of about 63 miles, 1 000 ft. wide through Philadelphia Harbor, 1 000 ft. wide at bends, except one quite sharp one at Bulkhead Bar which is to be 1 200 ft., and 800 ft. wide in straight portions. The original estimated cost of the project was $10 920 000, with $300 000 annually for maintenance. It may be noted that General Marshall, the then Chief of Engineers, U. S. Army, was of the opinion that the estimated cost of maintenance was far less than what he considered would be the result of actual work. Experience has demonstrated that General Marshall's opinion was well grounded. In 1922, a re-estimate of the cost of the completed work and that still remaining to be done on the original project was made, and the new figure, $15 300 000, was approved. While this amount is 50% greater than the estimate of 1910, it is noted that the higher cost of labor and materials since that date would more than account for this increase. The cost of maintenance, however, has increased beyond any difference chargeable to higher labor and material costs.

The District Engineer, Major Herbert Deakyne (now Brig.-Gen. Deakyne, Assistant Chief of Engineers, U. S. A., M. Am. Soc. C. E.), who submitted the report reviewed by the Board of Engineers for Rivers and Harbors, stated that past experience indicated that the maintenance of a deep channel by dredging alone would be unduly expensive, unscientific, and border on the impracticable. He added that "systematic contraction and control of the tidal flow are the proper means for the maintenance of the channel." And, further, "it is not expected that these controlling works will obviate the necessity of dredging a channel, but it is expected that they will simplify the problem of maintenance." He recommended construction of certain dikes for the purpose of contraction or controlling the currents.

The Board of Engineers in submitting the report on which the present project is based, agreed with the District Engineer in his recommendations.

The work on the 35-ft. channel necessary to obtain the first deepening from 30 ft. has been carried on practically without interruption since 1910, this original work having been done almost wholly under the contract method. The entire channel from deep water to Philadelphia has been completed with the exception of a 2-mile stretch in the upper end of Philadelphia Harbor, where dredging is now in progress, a few small rock areas in the channel near Chester, Pa., which it is hoped will be removed at an early date, and the removal of Mameluke Rock in the Upper Philadelphia Harbor, which is the chief obstacle to the completion of the work. About 90 000 cu. yd. of this ledge still remain, although it is noted that a 35-ft. channel of reduced width can be dredged to the westward of this area.

On the dikes recommended, the following work has been done (see Fig. 3):

Fisher Point Dike, completed to within 300 ft. of Petty Island. Full completion of this dike was prevented by action of citizens of New Jersey who objected to the cutting off of the channel between Petty Island and the New Jersey Shore.

Mifflin Dike, partly completed. Work on this dike was interrupted by the construction of Hog Island Shipyard and objections to the closing of the channel north of Tinicum Island.

Chester Island Dike, from the north end of Chester Island to the New Jersey shore, completed.

Oldman's Point Spur Dike, completed.

Bulkhead Bar Spur Dike, completed.

Edgemoor Dike, small portion finished.

Reedy Island Dike, partly completed. Work stopped due to protests of riparian owners.

Artificial Island, completed.

For maintenance, contracts are let for such work as is necessary in Philadelphia Harbor, and the dredging in the lower river is carried on by Government-owned and operated plant consisting of three sea-going hopper dredges and two pipe-line disposal dredges. At present (October, 1926), a navigable channel 35 ft. deep at mean low water and of widths varying from 200 ft. to the full width of the channel, extends from above the congested portion of Philadelphia Harbor to deep water in the Bay, except at a few isolated spots where the filling in of the channel has kept ahead of the dredging work. On the last examination made in August, 1926, the controlling depth on the center line of the channel was 32.7 ft., this, however, being at one point only.

Each year, however, shows an improvement in maintenance work and there is every reason to believe that with the erection of additional dikes as funds will permit for the combined purpose of disposal basins and control of the current, the maintenance of the channel at project depth can readily be held by dredging.

Methods of Work

As previously stated, the improvement on the Delaware River is a combination of dredging and dike construction. For the latter work the only features of note are that low-water dikes have proven ineffective, and that, due to ice and storm conditions and the nature of the bottom, these dikes must be of a considerably more substantial cross-section than was originally anticipated, this being still further accentuated when the dikes are also required to withstand thrust from back-fill.

Experience in dredging has shown the great advantage of the sea-going hopper dredge, particularly in maintenance work, over the stationary type. The critical feature in the dredging work is the disposal of the dredged material. Several methods have been tried. At first, the material was deposited in available locations in the river outside the channel. This was not found to be economical for the reasons that the bulk of the material quickly found its way back into the channel and had to be re-dredged; the available deposit areas were very limited due to the draft required for the dredges in going to and from the areas; and the filling of these areas frequently interfered with the

use of side channels for tugs, barges, and small freight carriers of shallower draft.

The method of agitation was then tried. This consisted in dredging by the sea-going dredges for long continuous periods, until the hoppers were filled with solid material, the lighter mud and sand being permitted to flow overboard. This was done only on ebb tide, with the expectation that the solids in the overflow would remain in suspension and be carried away by the tidal currents. This also proved uneconomical since these solids did not remain in suspension for any length of time, but came to rest in the channel below the point of operation, thus requiring re-dredging.

The method then adopted is that now in use, and consists in depositing the dredged material on fast land behind embankments or in the river in areas surrounded by dikes. The operation consists in dredging with the sea-going hopper dredges, generally between 30 and 60 min., until the economic load is obtained, this being determined from a consideration of material in hoppers, time of dredging, and time required to move to disposal basins. With bucket dredges the material is loaded into bottom-dump barges which are towed to the place of deposit. At the deposit basin, the dredgings from the sea-going dredges or the dump barges are deposited in basins dug in the bottom of the river and are then pumped into the prepared area by a suction pipe-line dredge. Incidentally, these areas, built up as described to 12 ft. and more above the plane of mean low water, may form valuable sites for industrial and other purposes.

The bucket dredge and the suction pipe-line dredge are familiar to all engineers. The sea-going hopper dredge is not so common. Essentially, it is a sea-going vessel, self-propelled, which contains, in addition to the quarters for the crew and the propelling machinery, suction pumps and bottom-dumping bins of a capacity depending on the size of the vessel. The suction pipe, so arranged as to permit raising and lowering, is fitted at its outer end with a drag head, consisting in general of a heavy casting with grated opening on the bottom. In operation, the suction pipe is lowered below the water level, the pump started, and the pipe then lowered until the drag head touches the bottom. As the dredge proceeds slowly over the area to be dredged, the material from the bottom, combined with a certain quantity of water, is pumped into the hoppers in the dredge. When the economic load has been obtained, the suction line is raised, the pump stopped, and the dredge proceeds under its own steam to the deposit basin, where the load is dumped. In brief, the sea-going hopper dredge is a combined pumping, towing, and dump-scow plant in one hull. Figs. 5 and 6 are views of two of these dredges.

The cost of the work done with the hopper dredge varies with the type of material and the distance from the deposit basins. For sand and gravel combined, with a comparatively long haul, the present cost is approximately 14.4 cents per cu. yd. of material removed. For soft material, with a comparatively short haul, the figure is 2.8 cents. The cost of pumping the material ashore by the pipe-line dredge varies from 2.5 to 10.8 cents per cu. yd., depending on the cost of construction of deposit basins and length of pipe line.

Depth by Contraction.

From a study of the work done under the various projects, and having in mind the studies of theoretical widths which have been discussed, the query naturally arises, why was greater use not made of the contraction system?

As a matter of fact the improvement of the river since the first systematic project for 26-ft. depth was started, has been by dredging supplemented at certain critical sections by longitudinal dikes or by spur-dikes.

Practically, however, the use of dikes has not proven so simple as a theoretical discussion of the problem would indicate, and the number constructed or projected are not in line with those that would be required under any conclusion based on such theoretical discussion, nor such as are desirable for the purpose of economic maintenance.

Some of the obstacles which have seemed and still seem unsurmountable are:

(*a*) Impracticability of placing such structures where they might not interfere with the use of existing or prospective commercial developments on shore, or where shore owners might not object to being cut off from deep water.
(*b*) Uncertainty under the present state of the science as to just where dikes should be located.
(*c*) The time required before any definite improvement throughout the entire length of stream could be obtained.
(*d*) The fixing of the channel at a certain depth.
(*e*) The increased difficulty of navigation of the channel by large vessels if it were developed on a series of curves instead of tangent ranges.
(*f*) Limited funds available.

A brief discussion of these obstacles may be of interest.

(*a*).—Since the sole reason for the improvement of waterways is to benefit and develop existing or reasonably expected future commerce, it follows directly that in the work of improvement such establishments or locations as do or may contribute to water-borne commerce should not be seriously interfered with. A case in point is the Harbor of Philadelphia extending for several miles along the river and so developed as to serve wharves and piers on both banks. In this case, a width of stream theoretically sufficient to give a certain depth is not the deciding factor. There must be ample room for harbor traffic both longitudinally and across the river. Anchorage grounds are necessary, and there must be sufficient width to permit ocean-going vessels to enter and leave their slips without too great interference with other vessels or risk of endangering structures on the other side of the stream. On the other hand, if the necessary width for this consideration is less or greater than the theoretical width for the desired depth, it is inconceivable that owners of existing wharves and bulkheads would re-arrange them to comply with lines based on this theoretical width. Structures in Philadelphia Harbor designed and located solely for reducing the stream to a theoretical width are, therefore, out of the question.

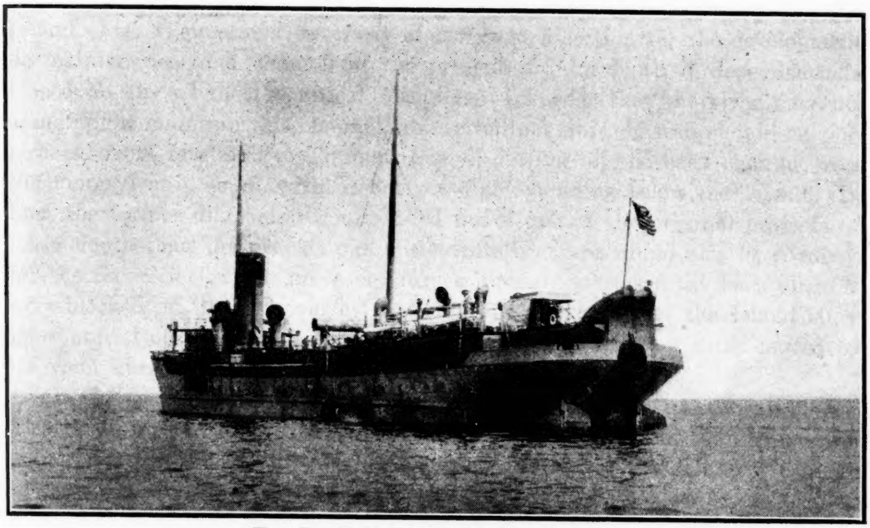


FIG. 5.—VIEW OF DREDGE *New Orleans.*

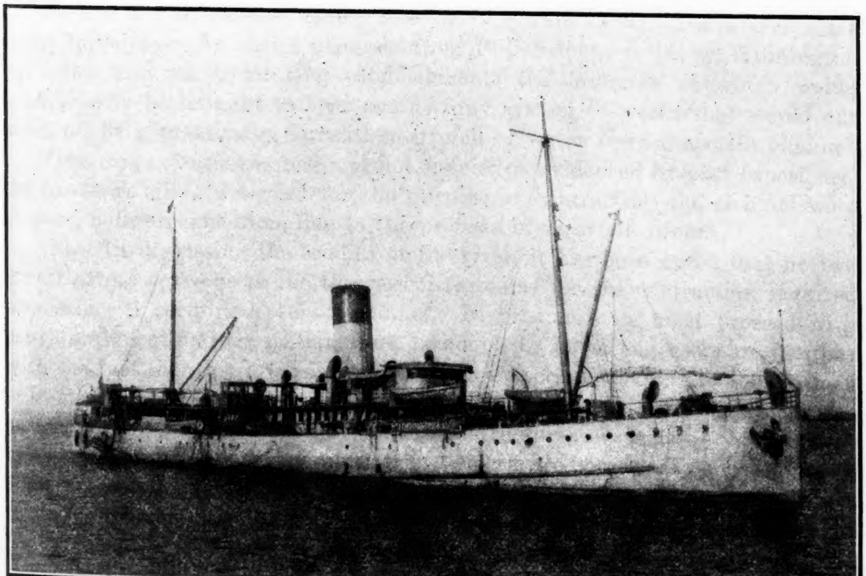


FIG. 6.—VIEW OF DREDGE *Manhattan.*

Pa

di
bo
ne
Isl
or
pla
alo
str
the
fac
les
for
wer
the
the

Ele
vid
ext
wat
yac
the
sho
it h
war

line
by
und
ther

At I
stop

inve
to p
caut
with
of a
exte
work
edge

limit
tem

It is admitted that this is an extreme case, and proponents of the training-dike system of improvement may claim that, while true for Philadelphia Harbor, it would not be true on the length of river below Philadelphia. The channels on the two sides of Tinicum Island and the stretch of the river below the Island (Fig. 3) present a waterway of too great a section for the development or maintenance of a channel of the present project depth if dependence is placed on any of the theoretical formulas. In order that the river currents alone might maintain this channel, a longitudinal training wall should be constructed from fast land to Tinicum Island, cutting off the back channel near the Pennsylvania shore, with probably a dike extending below the Island. In fact, the former dike was recommended under one of the original projects of lesser depth than the present one. Incidentally, there would also be required for the theoretical width necessary for maintenance in case the back channel were blocked off, the removal by dredging of a large part of the Island since the material of which it is formed is too heavy and compact to be moved by the river currents.

On this back channel is located one of the large plants of the Westinghouse Electric Company, and, while this industry might consent to the change providing it owns the riparian rights and that portion of the Island within an extension of its property lines, as this would permit it to have access to deep water, yet it is extremely doubtful whether the Town of Essington and the yacht clubs just below would be so complaisant. Likewise, one might imagine the attitude of the Baldwin Locomotive Works, located on the Pennsylvania shore just below Tinicum Island, if it were to find its present wharf, to which it has dredged a channel at its own expense, left several hundred feet landward of a training dike.

This is not an isolated case. The Pennsylvania and Delaware shores are lined for almost the entire distance from Philadelphia to below Wilmington by cities and manufacturing establishments the influence of which would undoubtedly be brought to bear against any system of works that would cut them off by a practically currentless stretch of water from the main channel.

That this objection is real and not fancied is evidenced by past experience. At least two dikes designed for the purpose of contracting the channel were stopped before completion, due to the protests of riparian owners.

(*b*).—In discussing the studies of the river, it has been noted that no two investigators agree as to the theoretical formulas for the contraction required to produce a certain depth of channel. In practice, one must proceed with caution, the cut and try method must be adopted. River engineers are familiar with the fact that any artificial work may cause serious changes in the regimen of a stream above and below the structure in question, this effect sometimes extending for long distances. Temporary structures or parts of the proposed work must be built and time must then be permitted to elapse before a knowledge of results sufficiently definite to warrant further progress can be obtained.

(*c*).—Due to the necessity of proceeding slowly and also on account of limitation in available funds, many years would elapse before a complete system of dikes could be built. In the meantime, while certain sections of the

river might be deepened to the desired draft, others would be untouched. The navigability of a channel depends on the governing depth of that channel. There might be a 35-ft. depth throughout 59 of the 60 miles of waterway from Philadelphia to the sea, but a depth of 20 or 25 ft. for the remaining mile or even a small fraction of it, would limit the size of vessels capable of navigating the channel to what could be carried over this governing section. In other words, there would be little real practical improvement until the entire system was completed.

(*d*).—Assuming longitudinal dikes could be placed with full assurance that the pre-desired depth could be obtained, there is the risk of stabilizing that depth, or of being compelled to construct a new series of contracting works to obtain a greater depth. Suppose, for example, the first project depth for the Delaware River had been obtained by the construction of longitudinal dikes. It is problematic whether funds would have been forthcoming for the 30-ft., and then the 35-ft. channel, if dependence had been placed on contraction works alone, for it must be remembered that theoretically the channel would develop close to the dikes along curved sections and the cost of constructing new dikes in the deeper waters would be much greater than the original construction in comparatively shallow water.

(*e*).—It is a fact that a sinuous channel could be navigated, but it is equally true that such a channel would be much more difficult for the handling of large boats than one laid out as a series of straight stretches for which easily distinguishable range signals may be erected, with short turns marked by light buoys or some other distinctive marks. The dangers of collision or of grounding in night running on a 100-mile channel, such as that first mentioned, would almost prohibit its use during hours of darkness, and would tend to diminish its usefulness even during daylight hours. The tendency of the channel to deepen on the outer part of the curve and hence near the contraction works would also increase the danger of navigation.

(*f*).—The revenues of even the Federal Government are limited. These revenues must cover a multitude of necessary Governmental expenditures. The result is that the amount available for all the river and harbor work throughout the United States is and always has been comparatively small. Shall these funds be used for works which after a lapse of many years may ultimately be beneficial, but which give little if any relief in the meantime; or shall they be used in such a way as to maintain what is already available and also give a progressively deeper channel, thereby permitting a gradual but steady increase in the size of cargo carriers? It would seem that the answer is obvious.

The writer has attempted to touch very briefly the obstacles to a complete control of a stream like the Delaware River by a system of dikes alone. He does not want to be misunderstood and be set down as opposing all use of dikes for partial control. That these works will not produce the desired deepening of a channel in the Delaware River in any reasonable time has been proven on that river by past experience, for in every case where they have been constructed, it has been necessary to expedite the action of the current by recourse to dredging. Past experience has been equally positive in proving that prop-

erly located dikes will assist materially in maintaining the channel. One principle of location of these dikes has been fully exemplified, that is, that they should be so placed that the ebb and the flood tides shall be forced into the same channel. Wherever this has been done the benefits, as regards maintenance, have been very apparent.

There are many places on the river where work of this kind could be done without injury to commerce or to riparian owners, and with great benefit to the cost of maintenance, and ultimately these works will be constructed. In the meantime, dredging must be carried on continuously to maintain the depth of channel already obtained, and basins must be constructed in which to place the material dredged. In this connection, it is again noted that the dikes and bulkheads constructed for these basins are designed and located not only for this purpose but with a view to their effect on controlling the currents which it is anticipated will result in a material decrease in maintenance dredging.

As is generally true, there is a happy medium between extremes which all things considered gives the best results, and the Government engineers have attempted to strike this medium, between the extremes of "all dikes with no dredging" and "all dredging with no dikes," in so far as funds and the legitimate demands of commerce for early improvement and continuous maintenance of increased depths will permit.

Economics

A paper on the Delaware River would be only partly complete without some reference to the commercial and economic side of the question.

The Port of Philadelphia includes the City of Philadelphia with its wharves on the Delaware and Schuylkill Rivers, Camden, N. J., Gloucester, N. J., and some minor points north and south of Philadelphia. This port serves a large area. For Eastern Pennsylvania, Southern and Central New Jersey and Delaware, involving a population of about 5 000 000, rail freight rates to Philadelphia are equal to or lower than similar rates to any other first-class seaport. It also has competitive rates with other ports for a large area, including Western Pennsylvania, Ohio, Indiana, Illinois, Michigan, Wisconson, Iowa, Minnesota, and the Dakotas.

By far the largest percentage of the water-borne tonnage is handled over the wharves of the City of Philadelphia. Practically all these wharves on the Delaware River are served by a belt line which permits shipments or receipts by or from any of the three railroad companies serving the community, the Pennsylvania, the Reading, and the Baltimore and Ohio. The Delaware waterfront is also paralleled by a wide avenue permitting easy trucking between piers and to cross streets leading to all parts of the city. On the Schuylkill River, while there is no belt line, a large majority of the wharves have switch connections with one or more of the railroads mentioned.

These railroad companies have large water terminal developments, including coal and ore piers with mechanical unloaders, general cargo piers, and grain elevators. Of the latter, one has a capacity of 1 250 000 bushels, another of 2 250 000, and a third is under construction, the first unit of which, with a capacity of 2 500 000 bushels, will be in operation at an early date.

The City of Philadelphia has for many years followed the policy of development of its water-front. Within the past 17 years, thirteen modern piers have been constructed by the municipality. Figs. 7 and 8 show cross-sections of two of the latest types of pier.

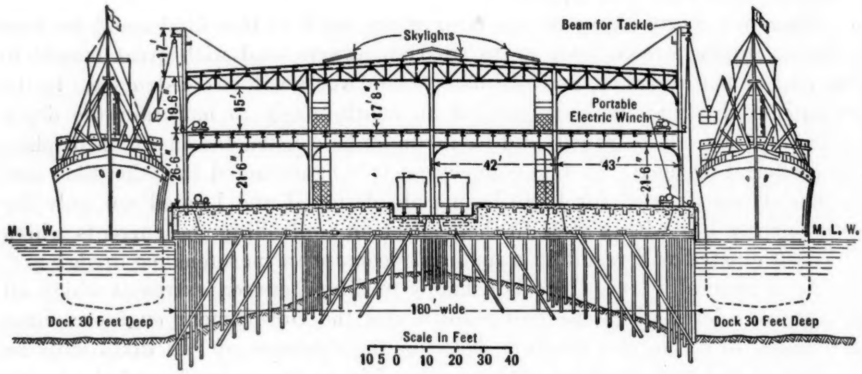


FIG. 7.—TYPICAL CROSS-SECTION, SOUTHWARK, PIERS, PHILADELPHIA, PA.

Aside from the rail terminals and the city-built piers, there is little permanent wharf equipment for handling ship's freight, dependence being placed in general on the cargo booms and winches of the vessels themselves.

Under the policy adopted by the City, none of its municipally owned wharves is operated by it, but is leased to shipping lines or ship brokers. The writer regrets that at least a few of these facilities have not been reserved for municipal operation, where all comers could be sure of obtaining berths and reasonable loading, unloading, and storage rates, although it may be added that these rates as charged at privately owned or leased wharves compare favorably with similar rates at other ports.

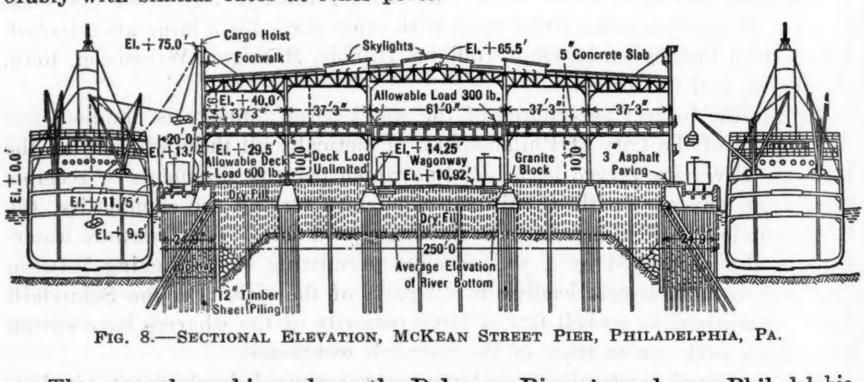


FIG. 8.—SECTIONAL ELEVATION, MCKEAN STREET PIER, PHILADELPHIA, PA.

There are three shipyards on the Delaware River in and near Philadelphia, which can construct or repair any vessel that can navigate the channel.

A large part of the inbound tonnage in the Port of Philadelphia consists of crude oil and oil products. Oil refineries and tank farms are located on the Schuylkill River and on the Delaware River at Marcus Hook, Pa., with one

large plant on the New Jersey side about 10 miles south of Camden and one just north of that city.

In addition to this commercial development, the Delaware River channel serves the League Island Navy Yard, which, in addition to being the largest Navy Yard in the United States, is the only fresh-water yard and also ranks favorably with any other similar commercial development in the country. The value of this yard in time of emergency, protected as it is by its distance from the ocean, is incalculable, and alone would warrant a large annual expenditure on a channel connecting it with the sea.

Table 5 gives a summary of the tonnage handled across the piers and wharves in the City of Philadelphia. This is taken from the reports of the Chief of Engineers, U. S. Army, for 1925, the latest printed report, and covers commerce for the calendar year 1924.

Table 5 covers only such tonnage as actually passed over the wharves and piers of Philadelphia. It does not include approximately 5 000 000 tons in transit, that is, tonnage in vessels touching at Philadelphia, but destined for other ports, nor does it include 4 000 000 tons of cargo carried on car ferries. The tonnages for Camden, N. J., and Wilmington, Del., are also excluded from this summary, although ships for these ports make use of the channel between Philadelphia and the sea throughout its entire length for the former and over approximately two-thirds of its length for the latter.

Every engineering work is subject to scrutiny as to whether or not it is a paying proposition. Examination of the Delaware River, from this point of view, will show that, the total cost of all new work on this river between Philadelphia and the sea, including the deep-draft channel for the lower six miles in the Schuylkill River, not only on the present project, but also on all former ones, for work done by the Federal Government, to June 30, 1925, was \$26 000 000 in round numbers.

Assuming an amortization of the first cost at the rate of \$1 000 000 yearly, and an annual interest of 4%, there will be during the next 26 years an average yearly cost to the United States of \$1 520 000. To this should be added the cost of maintenance.

The average yearly expenditure on maintenance work during the six years ending June 30, 1926, was \$1 475 000 for the Delaware River only. Under the law the maintenance dredging in the Schuylkill River must be done by the City of Philadelphia, until certain sewage disposal works are completed. It is impossible, therefore, to give annual maintenance figures, based on actual cost, but \$100 000 is an outside figure for this work. It is believed that these figures are fair ones for use in estimating the cost of maintenance for the next few years, and, in fact, are somewhat excessive, as they include the cost of disposal basins, which will be available for several years.

These amounts, interest on cost of new work and yearly maintenance, give \$3 095 000 as the amount the United States Government will be required to pay each year for the next 26 years on account of the 35-ft. channel from Philadelphia to the sea.

TABLE 5.—PORT OF PHILADELPHIA (THE DELAWARE AND SCHUYLKILL RIVERS), NET COMMERCE 1924.

Classes of commodities.	Foreign.				Domestic.							
	Imports.		Exports.		Coastwise.							
					Receipts.		Shipments.					
	Tons.	Value.	Tons.	Value.	Tons.	Value.	Tons.	Value.	Tons.	Value.	Tons	Value.
Animals and animal products	38 933	$ 12 122 637	11 272	$ 3 871 429	14 198	$ 12 911 941	11 555	$ 8 826 838	4 789	$ 7 953 564	80 747	$ 45 686 409
Vegetable food products	1 261 264	97 280 700	1 171 142	57 759 301	83 948	17 771 835	138 541	25 846 059	252 792	18 594 441	2 907 687	211 752 336
Other vegetable products	43 131	12 952 987	23 999	3 030 985	33 632	7 053 024	9 462	6 117 549	4 205	374 424	114 429	29 528 969
Textiles	76 562	38 255 512	11 366	2 683 283	86 709	87 630 945	40 655	32 868 396	....	....	215 292	161 438 136
Wood and paper	266 704	12 346 592	15 508	1 609 778	384 708	7 072 945	14 986	5 692 603	31 158	994 549	713 064	27 716 467
Non-metallic minerals	1 184 514	10 959 537	1 353 679	46 010 766	6 200 976	106 832 618	1 364 730	24 230 003	8 175 511	58 339 644	18 279 410	246 372 568
Ores, metals, and manufactures of	599 596	8 028 741	61 323	6 349 115	31 921	2 614 187	326 077	24 323 902	41 316	2 045 437	1 060 233	48 361 382
Machinery and vehicles	1 915	993 300	23 258	6 194 860	2 720	894 340	15 229	5 465 571	679	421 250	43 801	13 969 321
Chemicals	117 921	5 940 183	84 346	6 029 882	81 076	3 582 135	54 377	11 460 613	358 943	17 561 421	696 563	44 574 234
Unclassified	2 248	3 842 175	4 213	935 316	89 240	16 511 071	426 374	93 478 051	806 280	85 281 409	11 328 385	200 048 022
Total	3 592 788	$202 722 364	2 760 136	$134 474 715	7 009 128	$262 875 041	2 401 886	$237 809 585	9 675 673	$186 566 139	25 439 611	$1 024 447 844

What is the return on this yearly outlay? From Table 5, the figure of 25 000 000 tons is obtained as the total commerce passing over the wharves and piers in the City of Philadelphia. It would be erroneous, of course, to attribute this entire tonnage to the work done on the channel by the United States Government, since, before any work was started, there was an available depth of possibly 18 ft. at mean low water. The increase of tonnage due to increased depth as obtained by work done by the United States is impossible of exact determination, but it is believed a rough figure can be obtained.

Table 6 gives the number of vessel trips made during the year 1924, together with vessel drafts. The figures in Column (3) give the ratios between tonnage of cargo carried in a vessel of the draft given to that carried by a vessel drawing 18 ft. This is, as must be realized, quite arbitrary and may be far from the actual ratio. However, data were not available to determine the correct figures and the best obtainable were used.

TABLE 6.—Draft of Vessels and Number of Vessel Trips.

Draft of vessel, in feet. (1)	Number. (2)	Ratio of cargo to 18-ft. draft. (3)	Product. (4)	Total.
More than 30	18	5	90	......
28 to 30	129	4	516	......
26 to 28	512	3	1 536	......
24 to 26	363	2.75	998	......
22 to 24	476	2.33	1 110	......
20 to 22	664	1.75	1 162	......
18 to 20	1 336	1.25	1 670	7 082
16 to 18	1 500	1	1 500	......
Less than 16	5 798	0.75	4 348	5 848

Column (4) gives the product of Column (2) by Column (3). It is noted that, by using the arbitrary ratios adopted, the number of vessels of more than 18 ft. would carry 7 082 18-ft. vessel cargoes, while those under 18 ft. would carry 5 848. These vessels drawing more than 18 ft., therefore, carry about 55% of the yearly tonnage. Fifty-five per cent. of 25 000 000 tons gives 14 000 000 tons per year as the approximate increase of commerce due to the deepening of the channel by the Federal Government.

Using this tonnage and $3 000 000 as the annual cost to the United States, it is found that, for each ton the United States is expending 21.4 cents or, since the distance from Philadelphia to deep water in Delaware Bay is 60 miles, 3.6 mills per ton-mile.

While there is much controversy in regard to the advisability of improving inland rivers, the writer has yet to hear any arguments against the development of waterways leading to large seaports. The Port of Philadelphia is claimed by some writers to be the second seaport in the United States in the amount and value of its water-borne commerce. Whether or not actually second, it is beyond argument that this port ranks well up in the list of those following the Port of New York.

Foreign imports and exports to and from Philadelphia carry about the same rate as for New York and Baltimore. Rail freight rates to Philadelphia from these two cities for Class 6 commodities are 18½ and 17½ cents per 100 lb., respectively. Without its waterway the people of Philadelphia and its vicinity would be placed under a tremendous handicap in transferring imports and exports and its coastwise shipments by rail to or from either New York or Baltimore.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

TRAFFIC CONTROL BY ELECTRIC SIGNAL LIGHTS*

By M. O. Eldridge,† Assoc. M. Am. Soc. C. E.

It is generally conceded that the ultimate form of traffic control in metropolitan areas will be by means of electric traffic signals, and it is, therefore, pertinent that all engineers whose interests deal with traffic in any of its phases, give more than a passing thought to signals and their operation.

City planners, highway engineers, transit engineers, and public utility experts are interested equally with the engineer in finding the solution of the traffic problem. Although electric signals will not act as a cure-all they are, nevertheless, destined to be the dominating factor in reducing accidents and providing for the smooth and rapid flow of traffic.

One of the more difficult traffic centers of Washington, D. C., is Scott Circle (Fig. 1). The situation here is complicated because it is the intersection of six streets (three directions), the main traffic being north and south in Sixteenth Street. There is a considerable rotary as well as through movement, but signal lights have fairly well succeeded in solving the difficulties, as shown in Fig. 1. The essential features of this plan are the division of the circular street with an inner zone for through north and south traffic and an outer zone for rotary traffic; and the treatment of both diagonal avenues simultaneously as cross-streets.

Co-Ordinated System Improvements

If costly congestion is to be relieved the flow of traffic must be rapid. Rapidity of traffic is extremely hazardous unless it can be smoothly conducted with absolute safety at intersecting streets. Synchronized traffic signals, whereby all lights on a main artery of travel show "green" for a given period, then "red", were originally thought to produce the best results. Modern practice, however, has a tendency to abandon this form of control in favor of the progressive or co-ordinated system whereby a vehicle traveling on a

Note.—Written discussion on this paper will be closed in February, 1928.

* Presented at the meeting of the Highway Division, New York, N. Y., January 20, 1927.

† Director of Traffic, Dist., of Columbia, Washington, D. C.

main street gets a change of signal at each cross street and is then able to proceed the full length of the controlled section at a predetermined rate of speed without a stop.

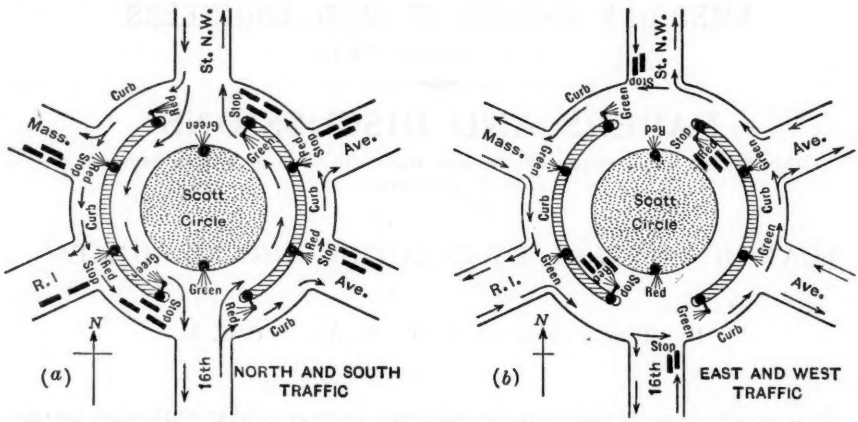


FIG. 1.—DETAIL OF TRAFFIC LIGHTS. CONTROL IN SCOTT CIRCLE, WASHINGTON, D. C.

SHORTER TIMING GIVES EVEN FLOW

Under the old synchronized system the timing on the main street was necessarily long in order that traffic might move as far as possible before a change of signals brought it to a halt. In congested business districts this tended to cause traffic jams in the cross streets where vehicles were necessarily stopped for 1½ and 2½ min. on each signal change, with the result that each cross street became completely filled. When the green light appeared there were too many vehicles in the block to clear the intersection before the red light came on again. In fact, it was not unusual for two or three changes of signals to be required to clear a given vehicle from a cross-town block.

The progressive and co-ordinated systems are operated on a much shorter timing, 30 to 45 sec. usually being sufficient. This results in fewer vehicles being caught in a block when the red light shows and more space being provided for parked vehicles to pull out from, or for others to get into, the curb. It also permits a smaller number of vehicles to clear the intersection on one signal change.

TIMING SET TO DESIRED SPEED

With the synchronized system no speed control was possible except by motorcycle police, whereas with this more modern type the timing can be set to permit any desired speed.

In Washington, a driver can run 12 000 ft. on Sixteenth Street where the traffic is comparable with that of Fifth Avenue, New York, in 7 min. at approximately 20 miles per hour. This speed can be varied by the simple expedient of changing the timing so that either a faster or slower speed will be necessary. It is obvious that in order to get through the controlled section, the speed must be maintained in conformity with the timing of the signals,

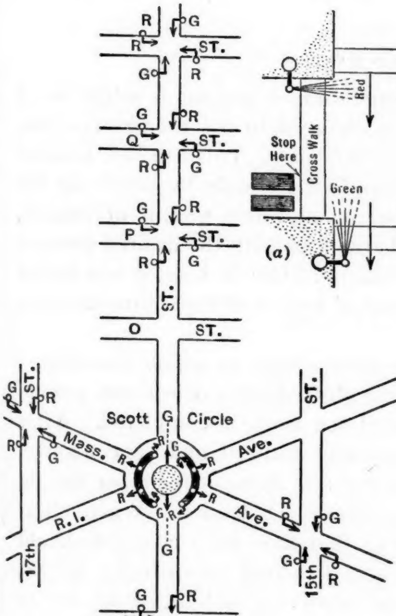


FIG. 2.—TRAFFIC LIGHT CONTROL, 16TH STREET AND MASSACHUSETTS AVENUE, WASHINGTON, D.C.

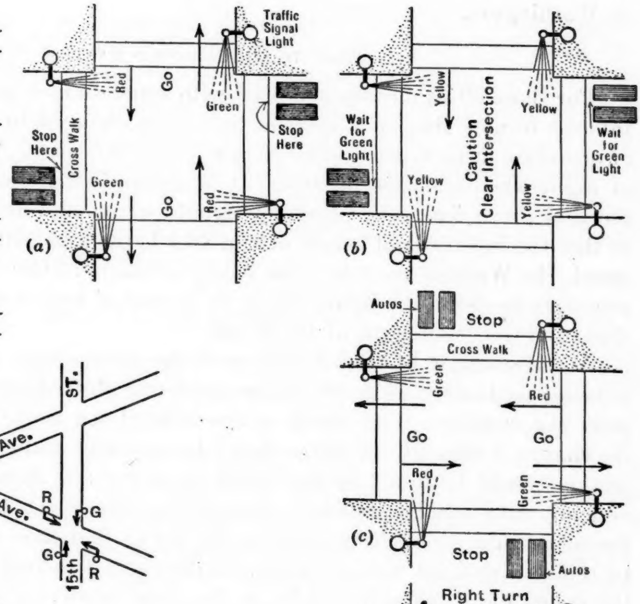


FIG. 3.—ARRANGEMENT AND USE OF TRAFFIC LIGHTS AT INTERSECTIONS, WASHINGTON, D. C.

otherwise the driver will be stopped at each intersection. Fig. 2 shows the operation of the progressive system as applied to a portion of Sixteenth Street, in Washington.

Easy to Change Old System

For those cities already equipped with synchronized systems it might be of interest to note that such systems are easily changed to the progressive type of operation at no expense other than a few hours' labor. This can best be done at night when traffic is lightest. It is accomplished simply by reversing the red and green lenses on alternate signals, or on alternate groups of signals, so that the interval and length of run may harmonize with the desired average speed. In Washington, where the blocks are 250 to 1 000 ft. long, it was found necessary to group the lights singly, or in sets of two or of three intersections, depending on the length of the blocks.

A traffic officer is needed to control the intersection at which the change is being made only while the lenses are being shifted; this officer can accompany the workmen from street to street until the work is completed. After the change is effected, the lights should be carefully watched, and the controlled section should be tested by an official car for a few days until proper timing and any additional signal-lens changes have been determined. When all is operating in a satisfactory manner, the lenses that have been changed should be replaced and the wire terminals in the signals shifted accordingly, so that the colors on all signals will be in the same relative positions, as an aid to color-blind drivers.

Amber Lights a Matter for Study

There is much difference of opinion as regards the use of an amber light in addition to the red and green lights. Experience in Washington, thus far, has led to the conclusion that the amber light is necessary in order that a maximum of safety may be afforded to the pedestrian and the intersection cleared of moving traffic. It is apparent, however, that the use of the amber light is a subject that merits the careful consideration of all traffic engineers.

The amber signal is not a "stop" signal; neither is it a "go" signal. Its purpose is to warn moving traffic that the "go" period is almost over and that a "stop" signal will be flashed in 5 sec. At a speed of 20 miles per hour, drivers who are within 50 or 60 ft. of an intersection have ample time to continue crossing the intersection and be out of the way before the direction of the traffic changes, and drivers who are farther back from the cross-walk have plenty of time to bring their cars to a gradual stop without suddenly jamming their brakes, and possibly causing the following vehicle to collide with them.

No Need of Forewarning for "Go" Signal

On the other hand, the use of the amber light following the "red" is not justified, because standing traffic has no need of a warning that the signal is about to change. Experience has shown that where this warning is given by flashing the amber light after a red light, standing traffic is likely to anticipate

the green light and start too soon, thus endangering the vehicles that have not finished clearing the intersection.

The ideal traffic cycle might possibly consist of a green light displayed for a predetermined number of seconds, followed by an amber light displayed for 5 sec., followed by a red light displayed for the desired length of time, followed by a green light, etc. Such a cycle would afford ample protection to pedestrians, as it would provide time for those walking in a green light zone to reach a point of safety, while the red signal on the intersecting street continued to hold traffic.

The use of green, red, and amber (yellow) signals in Washington, is illustrated in Fig. 3 (*a*), (*b*), and (*c*).

Right and Left-Hand Turns

There seems to be little uniformity in the method of making right and left-hand turns in different cities. Right-hand turns are always dangerous, because vehicles are obliged to pass over two cross-walks and necessarily cut through the line of pedestrians walking with the proper signal. In Washington, all right turns are made against the green light, in order that the turning vehicles will not encounter a line of moving traffic (Fig. 3(*d*)).

Left turns are less dangerous, because they can be controlled and the vehicle only passes over one cross-walk and at a time when no pedestrian has a right to be there. They should if possible be eliminated, however, at intersections where two-way traffic is permitted on both streets, as under this condition they tend to cause congestion.

The driver desiring to make a left turn in Washington (Fig. 3 (*c*)) pulls into the intersecting street when the light confronting him is green, keeping as far to the right as possible. He then stops alongside the cross-walk in such a manner as to block traffic on the intersecting street, leaving ample space for through traffic to pass on his left. When the signal changes, the turn is completed. This method of making the left turn on streets controlled by traffic lights has proven to be eminently satisfactory.

At points where a two-way street intersects a one-way street, the driver wishing to make a left-hand turn stops at the right-hand curb, just back of the cross-walk line, and when the signal changes to red, stopping through traffic on the two-way street, the turn is completed. Vehicles moving on the one-way street make the left-hand turn into the two-way street when running on a green signal.

Division into Zones

The signal system in Washington is divided into different zones, and the controlling apparatus in each zone is installed in a conveniently located fire engine house. At one of these controlling points an operator is continually on duty to operate the fire signal in that zone, and in case of emergency to throw the switch changing the control from automatic to manual, and then to operate the signal by the manual control while the emergency exists.

Signal Approach of Fire Apparatus

The Washington fire signal consists of an amber and red light displayed simultaneously in all directions (Fig. 3 (*f*)). This is a flashing signal operated at approximately 30 flashes per min., and all traffic is required to clear the intersections and to come to a standstill while the signal is on. The time that this signal is displayed depends on what fire apparatus is using the streets; it varies from 45 sec. to 3 min. The operators are furnished with a list of fire-alarm boxes, which call for a display of the signal, together with the time that the signal should be operated for each box. The Chief of the Fire Department reports that the use of this signal has been of great benefit to the movement of fire apparatus.

Electric traffic signals have come to stay. Their usefulness and efficiency have been clearly demonstrated, and there is no question that they are a vital factor in the reduction of traffic accidents.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

TRAFFIC CONTROL IN NEW YORK, N. Y.*

By Philip D. Hoyt,† Esq.

The chief elements of the traffic problem are matters for the Engineering Profession and the police. To a great extent the engineer's problem is one of the future, and the problem of the police is that of the present. While the engineer is planning overhead streets, underpasses, and measures for increasing the roadway capacity of existing streets, traffic is increasing at a pace so rapid as to threaten saturation before these projects can become realities. The police, in the meantime, have to keep traffic moving with the existing facilities.

The complexities of the problem in New York City are increased not merely in proportion to its great population, but also because of many other factors, some of them peculiar to the city. The topographical barriers that divide and surround it; the density of population; the diffusion of the centers of various activities over widely separated parts; the height of the buildings; and the extent of the suburban sections, all complicate the situation. More than 6 000 000 people live in New York City. The people who enter daily for business or amusement increase this population to 8 000 000. This visiting population affects traffic conditions far more than the resident population for the reason that those persons who come from outside the city use the city's streets, usually in the most congested centers and during the most congested hours.

The business population is largely concentrated in a small area of Manhattan. The streets in this area have undergone comparatively little change since the first automobile made its appearance—a period of about thirty years. The greatest change has come within the last ten years when automobile registration in the city has increased from 118 000 to nearly 600 000. There were 80 000 more automobiles in the city in 1926 than in 1925. During

---

Note.—Written discussion on this paper will be closed in **February, 1928**.

* Presented at the meeting of the Highway Division, New York, N. Y., January 20, 1927.

† Chairman, Traffic Board, Police Dept., New York, N. Y.

this period the registration in the suburban sections has increased just as rapidly—a factor which obviously has a direct effect on the volume of city traffic.

The opening of the Holland Tunnel to New Jersey and the completion of projected bridges across the Hudson and East Rivers will add to the congestion. The proposed operation of hundreds of buses from New Jersey and Long Island, and the use by the railroads and express companies of large fleets of trucks to carry merchandise into and through the city, is already planned. This great additional burden of traffic will be dumped over night into the city's most congested section before any adequate provision for it can be made.

The engineer must plan years ahead, and were it not for his activities in planning additional facilities, the task of moving traffic smoothly and safely would soon become hopeless. The widening of streets affords perhaps the quickest plan of relief, but the speeding up of traffic by the elimination of crossings where vehicles have to stop to let other vehicles pass can be accomplished only by elevated or depressed highways—a slower and costlier operation. The widening of highways and elevated roadways have been recognized as essential to the future needs of the city, and if these plans are not carried out, it will not be due to a lack of foresight on the part of the engineer, but to the difficulties of providing the necessary funds to complete them.

Parking

The problem of the police is to get the maximum use of existing facilities by traffic control. One form of control that has become a National problem is the restriction of parking. While the theory that the streets are primarily for the use of moving vehicles is undoubtedly sound, the assumption that moving vehicles exclusively should occupy street space may be subject to dispute. Lower Manhattan Island—that part between 59th Street, which is the center of congestion—illustrates the parking problem in its most difficult aspects. The normal day-time population of this area of slightly more than 8 sq. miles, is estimated at between 3 000 000 and 3 500 000, two-thirds of which come there daily for business purposes and leave at night. The principal business centers lie within this district. So general is the congestion there that it would be difficult to establish within it any special zones where parking prohibitions—not limitations—would be justified more than in others. Wholesale and retail business must have some access to its property. Indeed, most of those persons who chafe at the delays in moving through this congested area, have occasion to stop at some point in it, usually on the street.

The writer does not mean to imply that the parking privilege is not habitually abused, or that it is justifiable when it completely blocks traffic, as is sometimes the case, but merely to point out that in formulating a policy of parking restrictions, a medium must be found between one that considers moving vehicles only, regardless of how long they occupy the street, and a policy that permits halted vehicles to paralyze traffic.

Limited parking measures, however, are not so easily enforced. If the limit is a time limit, it is most difficult to enforce. Under a regulation that prohibits parking entirely any car that stops is obviously violating the law,

and if the driver attempts to leave it he may be told to move on. If drivers are allowed to park for a limited period, however, the police officers charged with the enforcement of the regulation must note the numbers of the cars and the time of their arrival, check up to see whether they are still there at the expiration of the time limit, and then find the drivers, who may be lost in some high building. Strict enforcement of such a regulation in the congested zone south of 59th Street, Manhattan, would require an additional force equal to the present total traffic force. A regulation of this character is being applied, however, with some measure of success to prevent flagrant abuses of the parking privilege in the congested section.

Another special parking regulation which has been invoked more successfully in Lower Manhattan limits parking on the main arteries to certain hours of the day. This rule grew out of an emergency regulation promulgated during a subway strike. A large number of buses were operated during the early days of the strike to carry people to and from business. Anticipating a serious traffic jam, the Police Commissioner prohibited parking on these arteries from 7:00 to 10:00 A. M., and from 4:00 to 7:00 P. M. The results of this regulation were so beneficial that it was made permanent with slight modification as to the hours, and is meeting with general satisfaction.

Another plan that is being considered is the limiting of parking to one side of a one-way street. Experiments with this plan have not progressed to a point where any final opinion as to its benefits can be expressed. It presents several disadvantages—one, the difficulty of determining which side of the street is to be favored, and, another, the packing of one side of the street with a solid line of parked cars, which prevents trucks or passenger vehicles from pulling up to the curb for loading or unloading.

One-Way Traffic

The experience of New York City with one-way streets has shown them to be of great benefit in congested areas—not only from the standpoint of traffic, but also of safety. The tendency of vehicles to keep in their proper lanes on one-way streets increases the capacity of the streets, and the elimination of complete turns removes one of the most frequent causes of traffic jams. One of the chief advantages of one-way streets, however, is that they remove the usual objection to left turns—the crossing of one or more lanes of moving vehicles..

It is not generally realized that 90% of the left turns made in the congested section of Manhattan are made without crossing a lane of moving traffic. The regulation compels left turns to be made when traffic on the avenues is halted. The vehicle that makes a left turn from an avenue into a one-way street, and the vehicle that makes a left turn from a one-way street into an avenue both have no traffic to cross, except the cars halted on the avenue. This not only expedites traffic, but removes one of the most frequent causes of accidents. From the standpoint of the pedestrian one-way streets are also very helpful as he needs look in only one direction when he

crosses. The one-way street is so well regarded by the police traffic officials that steps to double the one-way street zone in Manhattan have been taken.

Traffic Lights

The feature of traffic control which, in the opinion of officials of the Police Department, and the writer believes the public generally, has proved the most successful, is the traffic control light signal.

With the increase in the number of automobiles, the delays due to the crossing of traffic at the various street intersections became more marked. The chief cause of this delay was the lack of co-ordination between the police officers stationed at successive crossings on the same thoroughfare. A car would be stopped at one block to permit cross traffic to proceed, and a few seconds later would be halted again at the next important intersection because the officers at the two intersections were not working in unison.

As early as 1912 the officials in charge of traffic in New York began to make a study of measures to co-ordinate the movement of vehicles. The first test was made in Lower Broadway. An officer stationed at Worth Street and Broadway, which is situated on a slight rise so that he could be seen from Chambers Street on the south and from Canal Street on the north, gave a signal with a flag when he was ready to move cross traffic. The policemen north and south of him followed his signals at their crossings. This idea was abandoned after one day's trial on the theory that the volume of cross traffic varied at the different street intersections.

Experiments to synchronize the movement of cross traffic with semaphores were made in 1915. Each officer was instructed to watch the officer next to him and change the direction of traffic at the same time. The chief difficulty was that the time required to relay the signals over any considerable distance in successive stages was so great that the officers farthest away from the key signal were far behind with their signals. Imperfect as this plan was, it demonstrated that it was a step in the right direction. Later, an experiment was attempted to synchronize the semaphores by means of a signal flag at one central point. This was a little more successful.

The suggestion that the railroad block signals could be adapted to the control of vehicular traffic by means of elevated lights soon followed. Some of the older men in the Traffic Department were very skeptical of the advantages of such a system, and reports as late as 1917 are on file in the Police Department to the effect that such a plan had no merit. One of the objections was that the streets would soon become roadbeds over which the vehicles would speed like railway trains. However, the plan was tried and its success was instantaneous.

Wooden towers were installed on Fifth Avenue in 1918. Police officers were stationed in these towers to control traffic by means of lights. The towers were built high so that they would be visible one from the other, and the signals were relayed along the system by means of individual controls in each tower.

In 1923, the present bronze towers replaced the wooden ones. The old towers were removed from Fifth Avenue and placed at other points. Many additional towers were installed in various parts of the city in 1923 and 1924.

Extension of Light Control System

In 1924 the first system of pole signals was installed. This system had the advantages of not requiring the use of any street space and of making possible the operation of many signals over a wide distance from one central point. During 1926, 98 signals of the improved pole type were put in operation on 23 miles of streets, and contracts for 173 additional signals, covering 22 miles of roadway, were awarded. A further program covering more than 1 000 intersections on 66 miles of streets is planned for 1927. After this is completed traffic will be controlled mechanically at 2 243 intersections in the city, doing the work of 4 486 policemen at an annual salary of $13 215 000.

The installation of the traffic lights in this city is under the jurisdiction of the Department of Plant and Structures the engineers of which lay out the systems. The Police Department establishes the locations and plans other details relating primarily to traffic.

The Police Commissioner in making his latest request for an appropriation to extend the traffic lights, pointed out that in the congested areas of the city every cross street was a danger point both to pedestrians and occupants of vehicles. Most of these streets are thickly populated and thousands of pedestrians have to cross them daily. The careful driver, without this traffic control, must proceed slowly and cautiously, not only to be on the lookout for pedestrians, but also to watch for other machines issuing suddenly from side streets. The accident toll on many of these streets has been heavy. Many of the important intersections are controlled by police officers, but to provide them at all the crossings covered or proposed to be covered by mechanical means would be out of the question because of the great cost.

The experience of the traffic officials with control light signals has led them to plan the new systems in zones. If lights are installed on one of several congested and adjoining avenues, the congestion and hazards on the adjoining avenues without light control signals are greatly increased. To avoid the light-controlled streets, drivers will use the streets that have no lights and, unrestrained by any traffic control except at the most important intersections where police officers are stationed, speed along without regard to pedestrians or other machines attempting to cross their path. It is deemed better to complete the installation of lights on several adjoining avenues for a shorter distance than to place lights the entire length of one avenue to the exclusion of those adjoining it.

Problems of Light Control

The first traffic light systems in New York consisted of three lights of different colors, red, green, and white. This was later supplanted by a system of two lights—green to indicate "go" and red to indicate "stop". The chief difficulty of this system was that the change of lights was so abrupt that no interval was given for clearance. The question of using an amber light as a cautionary light to give warning of changes was considered. Experience of other cities, however, demonstrated that on the amber light vehicles were

likely to proceed from all directions. In order to obviate an abrupt change, an interval of several seconds has been provided between the red and green lights, and *vice versa*. During this period of no light, traffic in all directions must halt. For several months the traffic officials have been experimenting with a system under which the red light will show in all four directions during the interval. This, it is believed, will prove to be the ideal system.

Several other minor problems relating to the installation and operation of traffic light systems are yet to be solved. One of them is the timing at important intersections. On many of the streets where traffic lights are installed, the volume of traffic on one intersecting street may be many times that on the majority of the others. The usual timing is 2 min. for the light-controlled street to 1 min. for the cross streets; but if this ratio is maintained at important intersections, where in many cases the traffic is equally divided between the light-controlled street and the intersecting street, cross traffic would be blocked. On the other hand, if the timing at the important intersection were based on an equal division of traffic—say, $1\frac{1}{2}$ min. for north and south, and $1\frac{1}{2}$ min. for east and west, it would be out of tune with the remainder of the system. That problem is not yet solved. In the meantime, the officers at some intersections have to disregard the lights to get their cross traffic through. The problem is particularly difficult where two avenues, both controlled by traffic lights over a considerable distance, intersect. The volume of traffic on the two avenues might be three times that on every intersecting street, except the one intersection where the two cross.

Another problem giving much concern is that of diagonal avenues. Broadway, for example, runs diagonally, crossing every avenue from Fifth to Eleventh. In other words, Broadway is both an avenue and a cross street. At every point where Broadway and another avenue intersect, both are intersected by an important cross street. The timing on Broadway, as well as the other avenues which it intersects, is 2 min. for the avenue and 1 min. for cross streets.

Consider the intersection of Broadway, Sixth Avenue, and 34th Street: The last named has 1-min. periods in which to move traffic and halts of 2 min., while the traffic on the avenues is running. The 2 min. have to be shared by Broadway and Sixth Avenue—so that they have only 1 min. each. This requires a complete break in the plan of synchronization on Broadway and Sixth Avenue. Further, it requires at each intersection of Broadway with an avenue a special type of traffic light installation. Thus far no satisfactory solution of this problem has been found.

The operation of the progressive or wave system of traffic control lights such as that on 16th Street, Washington, D. C., has been studied by the traffic officials of the Police Department. The chief obstacle to its use in New York is the fact that most of the streets for which traffic control lights are needed have car lines. It is obviously impossible to maintain a uniform and sufficiently high rate of speed when two lanes of vehicles which are given the right of way by the lights are continually compelled to slow up or to stop whenever the street cars do so. However, it is hoped that proper conditions for experiments with the progressive system may be found.

The progressive principle, however, has been applied with considerable success to cross-town traffic. The average time required to run cross-town from one avenue to another is about 1 min. If the light signals on all the avenues are synchronized with respect to each other, the average car will get the signal to proceed at one avenue and arrive at the next one just as cross-town traffic is being shut off. This means a 2-min. wait at each successive avenue.

By timing the different systems so that an avenue clears cross traffic a minute later than the one west of it, vehicles going east arrive at each avenue just as it is closing for cross traffic. So that, in the zone where this system is now in effect, the vehicle can cover the distance in 3 min. with only a few seconds' stop, if any, whereas if all the lights were synchronized, it would be moving 3 min. and have 2-min. waits at each of three avenues—a total of 9 min. This is a reduction of 66⅔% in running time across this zone.

This system does not work the same in both directions, for the reason that the periods given for north and south and east and west traffic are not equal. The waiting time at each avenue for west-bound traffic, however, is reduced from 2 min. to 1 min., so that the running time for west-bound traffic is reduced from 9 min. to 6 min. or 33⅓ per cent.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

SNOW-REMOVAL METHODS IN NEW YORK, N. Y.*

By Elmer C. Goodwin,† Assoc. M. Am. Soc. C. E.

Of the difficult problems entrusted to city officials, authorities rank those of the Department of Street Cleaning and its snow removal work, second, the Police Department holding first rank.

The Department of Street Cleaning has to contend with forces, particularly during the winter season, with snow storms, gales, abnormal tides, and fogs, which cause frequent emergency conditions on land and water that render its work exceedingly difficult.

Speed is the main essential in snow removal and traffic is the overshadowing factor in preventing it. The temporary paralysis caused by a stagnant traffic creates a grave fire hazard. Upon the free movement of traffic depends the delivery of coal, foodstuffs, every form of mechandise, and the transportation of the public on its various missions. Therefore, the slightest delay in the removal of snow from the main arteries of traffic, results in serious traffic congestions which are very costly to business and to the public in general. It has been estimated, that a tie-up of traffic resulting from a serious snow storm, costs the business interests of New York City approximately $5 000 000 per day.

It is imperative, therefore, that a trained force and proper equipment be set in motion, at the earliest possible moment, to keep the roadways clear, and by keeping abreast with the storm, prevent the condition responsible for the congestion of traffic.

Traffic has reached such proportions that with a 5-in. storm, a great many streets are not wide enough to provide room for the piled snow and permit traffic to pass at normal speed. Again, speed enters into the problem for, on the rapid removal of the piled snow, difficult as the conditions are, depends the resumption of traffic to normal.

Note.—Written discussion on this paper will be closed in February, 1928.
* Presented at the meeting of the Highway Division, New York, N. Y., January 20, 1927.
† Examining Engr., Dept. of Street Cleaning, New York, N. Y.

In round figures the area scheduled for snow removal in the Boroughs of Manhattan, The Bronx, and Brooklyn, where the Department functions, amounts to 36 000 000 sq. yd., equivalent to 1 027 miles of 60-ft. streets. The street cleaning and snow removal work in the Boroughs of Queens and Richmond are under the supervision of the Borough Presidents of the respective Boroughs. Of the total area mentioned previously, 2 785 000 sq. yd. are cleaned by the railroads, equivalent to 79.13 miles of 60-ft. streets, leaving 33 215 000 sq. yd. for the Street Cleaning Department to handle. This area is cleaned partly by departmental forces and partly by contractors' forces. The area cleaned by the contractor varies, depending on how many streets of the schedule are assigned to him for snow removal. For a fall of less than 3 in., none may be assigned to him. In the case of a heavy fall the complete contract schedule may be assigned to him.

With the installation of modern methods and modern equipment for snow removal, excellent work has been accomplished in the clearing of the streets quickly after each storm. To do this, three large organizations have been formed, namely:

1.—The Snow-Fighting Force consisting of 677 Department motor vehicles, with plows attached, assigned to designated areas, 5 500 sweepers and drivers, and 10 000 laborers.

2.—The Snow Removal Force, consisting of (*a*) departmental equipment and labor, and (*b*) contractors' equipment and labor.

3.—The Street Railway Force, consisting of approximately 600 pieces of equipment and 1 500 laborers.

The first or plow organization of 677 pieces of motor equipment in readiness for operation with the approach of a storm, at a given signal, reports to its previously assigned routes, and starts the work of snow-fighting. Within 2 hours after the signal is given, this entire organization is in full operation, pushing snow from the center of the roadway to the curb. The day's work of 24 hours is divided into two periods of 12 hours each, which operation continues over the variously assigned routes until the roadways are completely cleared of snow. The plows work in teams of two machines, each covering areas of ½ lin. mile to 4 lin. miles of roadway, depending on the traffic and other conditions in each locality.

Immediately following the issuing of the orders bringing into operation the plow organization, further orders are issued to assign the 10 000 emergency snow laborers to the cleaning of cross-walks, and the piling and "sewering" of snow. These men are enrolled and assigned for work at 115 section stations throughout the city. They are furnished with the necessary equipment, such as shovels, scrapers, picks, etc., and are assigned in squads to definite areas under the direction of a squad leader who is a sweeper of the Street Cleaning Department. This sweeper has been previously trained in the piling of snow, and the disposing of it in sewers on the particular street to which the squad is assigned for work.

To supplement the cleaning of cross-walks by manual labor, the Department has had under observation a four-wheel tractor equipped with a straight-blade front-push plow. The performance of this tractor was so satisfactory

that the Department has purchased two cross-walk tractors for further observation, before the final adoption of this type of equipment as a standard.

The second, or snow removal force, is composed of three units:

1.—The Departmental Equipment and Labor.—This unit removes all the snow in Manhattan from 14th Street to the Battery, and, in addition, helps to clear the main arteries of the piled snow. In Manhattan, these avenues are Fifth, Sixth, and Seventh. There are also the four large bridges over the East River which, in addition to a considerable number of smaller bridges throughout the city, are handled entirely by Departmental labor and equipment.

This snow emergency work is carried on by the Department in addition to its regular function, which is the collection and disposal of the city's wastes, an average volume of 36 600 cu. yd. of ashes, garbage, and rubbish per day.

In 1926, the equipment available for hauling snow consisted of 900 horse-drawn dump carts and trucks of a total capacity of 1 800 cu. yd., and 1 100 automobile trucks ranging from 2 to 7½ tons, with body capacities from 3 to 14-cu. yd. water-level measurement, of a total capacity of 10 316 cu. yd., and a combined capacity for both types of equipment of 12 116 cu. yd. The Department also owns and has in operation 41 snow loaders of the conveyor type.

2.—Contractors' Force and Equipment.—The remaining area is divided into five snow removal districts. In general, they are District 2, Manhattan, comprising that area north of 14th Street and east of Fifth Avenue; District 3, Manhattan, north of 14th Street and west of Fifth Avenue; the Borough of The Bronx, which comprises in its entirety one snow removal district; the Borough of Brooklyn, which is subdivided into two snow removal districts; District 1, containing Street Cleaning Districts Nos. 1, 3, 4, 5, and 6, or, in general, the down-town section of Brooklyn. The remaining area containing six street cleaning districts, comprises the 2d Snow Removal District of Brooklyn.

The contract for the removal of snow in these districts is awarded, with the approval of the Board of Estimate and Apportionment, to the lowest bidder, at a public letting, held in October of each year. The contract specifies that the contractor in each district may be called out for snow removal work by the Commissioner after any snowfall, regardless of its depth.

It also specifies that the Contractor may start snow removal work of his own accord without the Commissioner's authorization, in snowfalls of 3 in. or greater. The contract further specifies that the Contractor shall remove the snow from certain mandatory streets which, in general, are the principal arteries of traffic. He may be assigned to the removal of snow in certain scheduled streets.

The total contractor forces start removal at 220 different points simultaneously, averaging 8 trucks, each of 4½ cu. yd. capacity per gang, making a total contractor's force of 1 760 trucks and 3 500 laborers. In recent years the contractor has employed, with considerable success, mechanical loading

devices, such as steam shovels, cranes equipped with buckets, and other devices in use in the general contracting field.

For supervising and checking this contract work, approximately 300 men are drafted from other City Departments.

All snow is disposed of through water-front dumps previously selected and assigned as such by the Dock Department, or at selected sewer manholes determined originally after a survey, consisting of 55 000 observations made of the flow in the various sewers. As a result of this survey, the sewers in the Boroughs of Manhattan, Brooklyn, and The Bronx, are divided into four classes, depending on the volume and velocity of the flow in the sewers. They are:

(*a*) Sewers that will take any quantity of snow that can be dumped into them.

(*b*) Sewers that will take snow that is shoveled or panned into them slowly.

(*c*) Sewers that will take snow panned or shoveled into them slowly, in addition to which water must be delivered from the hydrant by a hose.

(*d*) This class of sewer is unavailable for any kind of snow disposal work.

The sewer maintenance crew in each Borough is assigned to the work of keeping the sewers clear of obstructions and freely flowing.

3.—The Street Railway Organization.—Under the charters by which the street railway companies operate, they are bound to maintain and keep clear of snow, an area included between their tracks plus 2 ft. outside each outer rail.

In lieu of cleaning the area just mentioned, the Commissioner enters into an agreement with the various railway companies, for the cleaning of an area equivalent to the mandatory areas required to be cleaned by their charters.

Approximately, this area amounts to 3 000 000 sq. yd. The forces of the street railway companies are concentrated at certain prominent thoroughfares throughout the three Boroughs with their own equipment, consisting of flat cars, horse-drawn and motor-driven trucks, snow loaders, and labor.

The railway companies start their snow brooms to clear the tracks along their entire routes as soon as weather conditions indicate that there will be a continued storm. The Department and all snow removal organizations are dependent on the U. S. Weather Bureau for advance information concerning the probability and intensity of the coming storm. In addition, the Department maintains a set of self-recording meteorological instruments.

To handle the modern equipment which the department now employs in the disposal of snow, there has been established, originally under the supervision of the engineers, but now directed by the Superintendent of Equipment and Inspection, a school for the training of the department employees.

Upon the completion of the course, the employee is required to pass a test of the State License Bureau, and also qualify in an examination held by the Civil Service Commission, for the position of auto-truck driver. Three thou-

sand men have passed all tests and have been assigned to operating the motor apparatus.

Snow removal cost data vary, as snow has no fixed characteristics. A 5-in. storm followed by rain and thaw may be removed at a cost of $150 000, whereas a 3-in. storm with freezing weather and sleet, which would cover the streets with ice, followed by low temperatures, may cost $700 000. The average cost per year for snow removal in New York City, for the Boroughs of Manhattan, Brooklyn, and The Bronx, approximates $3 500 000.

The average snowfall for a season is considered to be 28 in. Further, the Department must be prepared for a season of heavy snowfalls, such as occurred during the winter of 1922-23, when more than 55 in. fell. During that winter 27 snow storms occurred, at intervals of about 2 per week. It is estimated that the volume of snow that fell on the streets scheduled for cleaning by the Department was 55 000 000 cu. yd.

Assuming that the weight of snow is 6 lb. per cu. ft., which is about the minimum, there were approximately 4 455 000 tons of snow to be disposed.

The first three storm days, December 7, 14, and 17, 1922, with a total fall of 3.5 in., were handled entirely by Department forces.

From December 28, 1922, to March 19, 1923, there were 24 individual storms coming so closely that it required 81 days of continuous work by the Department forces, the Contractor and the railroads.

During this period, the total area cleared by the Department forces was 81 446 000 sq. yd., and by the Contractor, 79 215 000 sq. yd., a total of 160 697 000 sq. yd.; in other words, the original area of 33 215 000 sq. yd. scheduled for removal by Department and contract work, was done over entirely, five times, during the period.

The cost of piling, contract work, and supervision was $2 031 200. The cost of the departmental work was $2 967 900, making a total of $4 999 100.

In conclusion, snow removal in New York City is based on the theory that when the snow begins to fall, plows and snow brooms will be started out to plow and sweep the snow from the center of the street, in ridges, to the side along the curb. At the same time crosswalk plows will clear the street intersections for pedestrian traffic.

During the snowfall, Department motor trucks, and after the snowfall, Department, contract, and railway trucks and loaders are started on the streets at important points, where the ridge is piled, loading the snow into the trucks in order to remove it while it is soft and easy to handle and before it is packed down by traffic. Sewering is the most economical method for the final disposition of snow.

The engineering force, in co-operation with the engineers of the Sewer Bureaus of the various Borough Presidents, are striving for the installation of sewers of adequate size with manholes conveniently spaced on the various thoroughfares, which would be adapted to the disposal of snow directly by plowing and dumping from trucks.

It is proposed to connect the water mains to the sewers directly so that when it becomes necessary to augment the flow, water sufficient for the purpose may be readily utilized.

It has been found that hand labor is non-productive to about 40% of the time, due to the constant stream of passing vehicles in the streets.

With this in mind, the Department is striving for the perfection of mechanical appliances as a substitute for hand labor, and it is safe to predict that within a few years, all snow removal work will be handled mechanically. The success of snow removal depends to a great extent upon the co-operation of the public.

With the public educated to the extent of providing their motor vehicles with skid chains at the first sign of a snowfall, and the prompt clearing of sidewalks, and the piling of snow at the curb, making a single mass of the piled and ridged snow in the streets, the problem will be greatly simplified.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

SNOW REMOVAL FROM STREETS IN BOSTON, MASSACHUSETTS*

By Edward F. Murphy,† Esq.

Prior to 1926, Boston, Mass., like many cities in the snowbound districts of the United States, was not properly equipped to prevent the economic loss which follows in the wake of severe winter storms. Mayor Nichols, in January, 1926, appointed a Board of Municipal Emergencies to investigate the advisability of the purchase by the City of proper equipment for snow-fighting. This Committee, composed of the heads of the principal Departments of the City, met from time to time and agreed on a definite policy regarding needed equipment. A loan for $400 000 for equipment adopted on the recommendation of this Committee was applied to the purchase of the following:

Twenty 7-ton White trucks, with blade-plow attachment.
Twenty 7-ton Mack trucks, " " " "
Ten 7-ton La France trucks, with blade-plow attachment.
Ten 7-ton General Motors Corporation trucks, with blade-plow attachment.
Three Barber-Green snow loaders.
One Holt tractor with plow attachment.
Three Mead-Morrison tractors, with plow attachment.
One wrecker for emergency.

The first snow storm for testing the new system occurred on December 5, 1926, shortly after the Department had received the equipment noted. This storm started about 11:00 A. M., and increased in volume very rapidly. The call for action was sounded, and as fast as possible all plowing equipment was started.

The work of assigning plowing trucks and tractors to certain plowing routes had been carefully mapped, and without delay the opening of all streets in certain areas was in progress. In the business, shopping, hotel, and theatrical districts, forty-five fast-moving truck plows were assigned to

Note.—Written discussion on this paper will be closed in February, 1928.
* Presented at the meeting of the Highway Division, New York, N. Y., January 20, 1927.
† Deputy Commr. of Public Works, Boston, Mass.

remove the snow to the sides of the streets. None of these plowing routes was more than a mile long, and the work accomplished during this 9-in. storm received the approval of the most critical.

The remaining fifteen large truck plows were assigned to other districts, and the tractors were assigned to the hill districts in which hospitals and similar institutions were situated. The plowing continued for the duration of the storm, and for some time after, to insure proper roadways for all motor vehicles. The work of snow removal began shortly after the plowing was started.

This snow-removal force consisted of:

1.—The total city forces, amounting to approximately 1 500 men, with 150 trucks and 140 horse-drawn vehicles to cart away the snow, and 4 mechanical snow loaders.

2.—Emergency labor to assist city forces, amounting to approximately 700 men.

3.—Snow-removal force composed of trucks and laborers employed by contractors, approximately, as follows, 150 trucks and 750 men.

4.—Street railway forces, as follows, 9 snow loaders; six 5-ton caterpillar tractors; 3 Walter truck snow fighters; 6 differential cars equipped with snow plows; 3 rail cars with side plows; 44 Champion snow plows on 5-ton trucks; 36 motor trucks, ranging in capacity from 1 ton to 7 tons; and 162 single car plows; and 16 sweepers.

It has been the custom in Boston for many years to advertise early each fall for competitive bids for snow removal in twelve districts, as the City forces are not large enough to do all the work. The city is divided into districts, and the prices range from 50 to 65 cents per cu. yd. for snow removed from the city streets. The following tabulation shows an average day's work in each contract district:

District.	Cubic Yards.
1	3 714
2	1 109
3	2 867
4	7 986
5	2 613
6	5 317
7	2 760
8	2 946
9	1 903
10	2 752
11	2 472
12	1 262
Total	37 700

This contract work is supervised by the engineers of the Public Works Department, and as most of the contractors work on two 8-hour shifts, it requires a very close watch to guard against any criticism.

The snow is dumped into large intercepting sewers and tidal water. All bridges over salt-water channels are provided with large openings and have been of great assistance in disposing of the snow from the city streets.

The official records of the U. S. Weather Bureau show that Boston is well within the heavy snow-falling area, and the records since 1918 are, as follows:

Year.	Inches.
1918	44.90
1919	15.80
1920	77.60
1921	32.80
1922	47.20
1923	55.90
1924	30.00
1925	21.20
1926	62.00

In December, 1926, about 24 in. of snowfall was recorded, and if it had not been for the excellent equipment the financial loss to business could not have been estimated.

There was no interruption of traffic, and the public was agreeably surprised the morning after the storm to find that all the snow had been removed from the business district.

The two snow storms that occurred in December, 1926, were of unusual intensity, one amounting to 12 in. in about 10 hours.

The
indi
that
on t
cho
Sur
a lo
mea
use

ing
two
mil
sph
cre
sha
an
Me
the
dis
it
su
th
to
of

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

BOUNDARY SURVEYS*

By C. T. Johnston,† M. Am. Soc. C. E.

Historical

Boundary surveys probably represent the oldest branch of applied science. The necessity for marking the boundaries between plots of land cultivated by individuals or families prompted men to resort to measurement. It is probable that an interest in geometry was stimulated as complicated figures were laid out on the ground. Geometry signified the measurement of the earth. Those who chose the name doubtless had little appreciation of its ultimate significance. Surveying was doubtless confined to the measurement of small areas of land for a long time. Men of inquiring minds gradually became interested in extensive measurements which were probably considered theoretical and, consequently, useless by the more "practical" elements of ancient society.

An ancient Sanskrit manuscript contains the following sentence: "According to the Chaldeans, four thousand steps of a camel make a mile, sixty-six and two-thirds miles a degree, from which the circumference of the earth is 24 000 miles." Pythagoras was the first known authority to declare the earth to be spherical, although Thaleas and Anaximander should be given almost equal credit. Erastosthenes (B. C. 276) noticed that about June 21 the sun cast no shadow in Southern Egypt, while near the Mediterranean Coast the sun made an angle with the vertical equal to one-fiftieth of a circumference. Since the Mediterranean shore is north of the intersection of the Tropic of Cancer and the Nile River, he estimated the circumference of the earth to be fifty times the distance between the two places. Although this was a very rough measurement, it stands for the first serious attempt to discover the shape of the earth. The sun's diameter was neglected by this early observer, and he wrongly assumed that his two observation stations were on the same meridian. It is impossible to analyze the errors which crept into his work, due to the fact that the length of the unit of measurement—the stadium—is not known.‡

Note.—Written discussion on this paper will be closed in **February, 1928.**

* Presented at the meeting of the Surveying and Mapping Division, January 20, 1927.

† Prof. of Geodesy and Surveying, Univ. of Michigan, Ann Arbor, Mich.

‡ "Elements of Geodesy," by Gore.

Cadastral Surveys

It is impossible to follow in detail the progress of cadastral surveying. That it laid the foundation for other branches of science there can be but little doubt. While society at large was not ready to accept the theories of these early investigators for nearly 2 000 years, there was doubtless always a small group that stood steadfastly in their defense. Copernicus and Galileo were brave publicity agents, who hastened the day when it would be safe to discuss such scientific questions openly. The surveyor again, in the early part of the Eighteenth Century, not only proved the earth to be spherical, but by the completion of triangulation systems in Peru and Lapland, under the auspices of the French Academy, demonstrated that it is flattened at the poles, thus justifying the mathematical calculations of Sir Isaac Newton.

The three great sub-divisions of boundary surveying are cadastral surveying, surveys of political boundaries, and surveys of riparian boundaries. Cadastral surveying is probably the oldest branch of applied science, and it opened the door for many other scientific interests. Any one who visits Egypt, India, or China, may see surveys in progress which are probably similar to those conducted by the early pioneers of this applied science. In most of this work wooden poles have always been used for making linear measurements. In many places in Egypt the farm areas are long and narrow. This facilitates cultivation to some extent and reduces the liability of serious error in surveying work. Monuments are generally placed along embankments or roads which are elevated above agricultural lands. Even if embankments or roads extending along the ends of farm plots may not be parallel, it is not difficult to find where monuments should be set to indicate the location of the boundaries. The area of a farm thus laid out may easily be computed by multiplying the average lengths of the sides by the vertical distance between them. If these farms were laid out in plots bounded by sides of practically equal lengths, native surveyors would encounter many difficulties largely eliminated under the system generally adopted.

Under English occupation (1882-1918), a triangulation system was extended from the Mediterranean to Assuan. There was something inspiring in this work. The ancient Egyptians had done much to lay the foundation for several branches of science. For hundreds of years they had performed their surveys and finally worked out a system which seems to have given general satisfaction. Near the beginning of the Twentieth Century another race entered the Valley of the Nile to provide a great network to which the cadastral surveys of ancient Egypt might be referred. If the ancestors of the English people existed at the time surveying was born in Egypt, they have left no record. The interesting point is that surveying and other sciences spread from countries now regarded as backward, to be improved, if not perfected, by more advanced races. It would seem that the representatives of Great Britain undertook the triangulation surveys in Egypt as an evidence of their appreciation of the achievements of the early Egyptian pioneers of science. Egypt bestowed a gift upon the early Mediterranean civilization. Hundreds of years

later the descendants of another branch of the human family returns the gift with interest, measured in terms of scientific growth.

It is not proposed to say much in this paper about surveying equipment. Improved equipment has been provided as rapidly as men have become qualified to analyze field work and make proper use of data. Men have designed and built better equipment as they have appreciated the necessity for accuracy. Instruments do not necessarily develop this appreciation. They partly satisfy the craving for accuracy. Cadastral surveying is an applied science regardless of the character of field or office equipment. The principles of surveying remain unchanged whether one uses a modern theodolite or a number of wooden poles. The principles of pure mathematics are the same whether the computer uses a pencil or pen. Probably pure science and surveying equipment have been developed to the point where cadastral surveys may be made to any reasonable degree of precision. Unfortunately, those adequately qualified in science have not frequently given their time and energy to cadastral surveying.

In the United States, Congress and administrative officers having authority over public land surveys might have obtained good advice from the Coast and Geodetic Survey, prior to the acceptance of any fixed plan or system. It was evidently not the intent of Congress to inject much science into public land surveys. For fifty years after the Constitution was ratified, lands were exploited by speculators who had a political advantage at Washington. It was impossible, until 1842, for a citizen of the United States, and a home-seeker, to purchase lands from his own Government. Under such conditions one should not anticipate the development of a scientific branch devoted to public land surveys. After most of the valuable lands were surveyed, and the opportunity for speculation had largely disappeared, public land surveys were placed on an approximate scientific basis. Because political representatives in past years failed to perform their full duty, those who represent science in surveying, are not relieved from responsibility at the present time.

Although this paper is not prepared for the purpose of advocating sweeping reforms, it will call attention to a few points which may disclose some of the causes for the apathy of the civil engineer toward the only branch of engineering known to the Father of our country. If engineers are to claim him they must accept surveying as a branch of civil engineering. If surveying is accepted as a branch of civil engineering, it must be regarded as a branch of applied science. There has been a tendency in many States to register so-called land surveyors separately from civil engineers. In many States, candidates applying for registration as surveyors are required to take examinations which many civil engineers would find difficult, if not impossible. Colleges and universities which teach surveying have been slow to introduce and require adequate preparatory training in the fundamental sciences. No one should be considered qualified to engage in cadastral surveying who can not make and compute a traverse. Many men who call themselves surveyors and who are presumed to do the work of surveyors are unable to complete a traverse survey. Many who know how to complete the field work and make the com-

putations do not know why they follow certain rules when they balance the traverse—dealing with errors arising from angular and linear measurements. Any one having a knowledge of probability and the method of least squares understands why these rules are followed. The average student of surveying can not select the best series from a number of series of direct measurements on a single quantity; in other words, if the average student, to say nothing of the practitioner, had before him several series of measurements of a distance between two points on the ground, he could not select the best series. It is unnecessary to go further in this direction because similar weaknesses may be found throughout most of the surveying courses now offered. The engineer cannot argue that it does not pay to apply science to cadastral surveys. He must accept the responsibility for accuracy in such surveys if he is to protect his professional reputation.

Political Boundaries

The surveyor has never been permitted to say very much about the location of boundaries. Freeholders generally agree between themselves as to the position of monuments, and it is the surveyor's duty to extend lines between them and to make computations and records. The surveyor is consulted even less in connection with political boundaries. The boundaries are usually described by public officers who are selected without much regard for their special abilities. A man who secures a place on a commission which is to define an international boundary might be called a statesman or diplomat. A few members of Congress in a committee room in Washington may agree on certain lines marked on a map as the boundaries between territories or States. The average citizen would probably classify these men as National legislators. A similar group in a committee room in some State Capitol may determine the boundary lines between counties. These men may be designated as State legislators. Another group of officers sitting at a county seat may describe the boundaries of townships. The latter class of boundaries are so unimportant at present that citizens pay little attention to them, and public officers engaged in such work are generally ignored. Political boundaries of these four types are inclined to become fixed and established. There is little growth. New nations, states, counties, and townships are not frequently organized. Although such lines when once established and monumented may become static and almost forgotten, there are political boundaries in the United States which display change in marked contrast. The boundaries of municipalities are constantly shifting. The United States can boast of one city whch embraces the largest area of land of any muncipality in the world. As municipalities grow, a few of the original monuments set in connection with public land surveys, or possibly related to political boundaries, are respected and protected.

Statesmen, politicians, or whatever they may be, describe boundaries for areas that lack boundaries. This generally means that little surveying has been performed in the locality under study, or it may mean that all parties concerned in the establishment of boundaries have agreed to place lines where they will result in the least damage, such as on the summits of mountain ranges and near the middles of lakes and rivers. Sometimes these public

servants find lines on a map which seem to be governed by definite rules. These are generally astronomical lines which may be drawn with a reasonable degree of accuracy. Whether the map built around them has many of the elements of accuracy is another matter. Many States have accepted astronomical lines for boundaries wholly or in part. One State has a portion of an arc of a circular curve as a part of its boundary. It probably never occurs to those who accept astronomical lines as boundaries that they are not commonly marked on the ground. Before many years farmers and other freeholders are asking public servants to inform them as to the county or State in which their lands are located. Sometimes adjustments are made in boundary lines so as to refer them to public land surveys rather than to astronomical lines. Sometimes the surveyor is asked to locate such boundaries in order that lands lying near them may be justly taxed. Diplomats, or statesmen, frequently locate the starting points for boundary lines on some almost inaccessible mountain or in deep waters, because these are prominent and well-known features of local geography. As a matter of course, the surveyor who performs field work in connection with boundaries so described is inclined to be critical of such diplomats or statesmen.

The political, social, and economic interests of nations, states, and municipalities will doubtless always outweigh scientific considerations which might be applied to boundaries. The surveyor, who is the only exponent of science in connection with the establishment of these boundaries, must rest satisfied when he obtains a reasonable interest in good surveying practice. The public cannot be expected to understand him when he advises control surveys. He may use calculus, the theory of probability, and guides from many other scientific fields to prove his conclusions. He cannot discuss residual errors with officers clothed with power authorizing them to describe boundaries in their own language. If surveyors of 100 years ago had been economists they might have shown Congress that poor surveys are always unprofitable. Any exacting application of science requires time, and time is always related to money. Most Governments spend large sums in support of commissions which define boundaries and supervise public surveys. The general public probably does not realize that special ability is available when descriptions of political boundaries are to be prepared. Expert help of this kind has been on the payroll of the Federal Government for more than 100 years. In no case known to the writer has one of these scientists been appointed to a commission or board which was to discuss or settle a political boundary. State Governments follow the same general policy. City boundaries are extended to harmonize with the financial interests of real estate speculators rather than to protect local public interests.

Regardless of this evident handicap, cities are making progress in connection with the description and location of their boundaries. Regulations regarding the survey and plotting of city subdivisions are improving from year to year. The stimulus which brings about these changes, does not come from engineers or surveyors to any great extent. Municipal authorities are showing a different attitude toward the plotting of areas to be incorporated within town or city limits.

The influence of the landscape designer is beginning to be felt. It may be that art and science will go hand in hand in the future development of American cities. These influences may dominate as the public discovers that they guarantee the only lasting benefits a municipality may enjoy. Society will be best served when science is permitted to extend all its aids to the description and location of political boundaries. Some record should be made of the splendid service already rendered by able scientists who have sacrificed much to mark on the ground political boundaries that have been inadequately described as a result of a compromise between contending forces.

Riparian Boundaries

As a result of natural laws, riparian boundaries fall into a distinct subclassification. The nations of the world have probably not yet reached a final conclusion as to their attitude toward riparian boundaries along the oceans and great seas. There is a general understanding that nations have control of waters for a distance of three miles off their shores. That this limit is not one that will always prevail is suggested in times of war and when new administrative questions arise.

The surveyor must maintain an interest in riparian boundaries regardless of how much the lawyer may complicate descriptions thereof. The surveyor must locate them on the ground if they are ever to be located. Riparian boundaries, together with riparian rights and riparian wrongs, are matters which cannot be analyzed in detail at present. Authorities in the common law relating to riparian boundaries admit that it is unstable and cannot be generally applied, but they assume that it is growing. When the surveyor appeals to the lawyer for specific advice, he is likely to be given something that cannot be applied in the field, or he is told that the matter has not been definitely settled. The surveyor often wonders whether Congress and administrative officers had any purpose in view when official regulations were issued regarding the meandering of lakes and rivers in connection with public land surveys. Entrymen under public land laws certainly secured title to no land beyond the meander survey in so far as the laws of Congress and the rulings and regulations of administrative officers are concerned. The Courts, under the common law, have permitted the freeholder to claim title to lands beyond the limits of the meander survey. Most of the complications which face lawyers and surveyors alike, may be traced to the absence of any definite boundary that may be described or established after the meander survey is once passed. No definite limit having been found, the surveyor is recommended to carry his lines to threads of streams, and to the middle of lakes. The surveyor who knows something about threads knows that streams do not have them. Although matters of this kind have been discussed for many years, they have had no practical application. A boundary, to be of any service, must be susceptible of location on the ground.

Riparian boundaries must be made tangible. Lands and waters can be measured. Science is willing to shoulder responsibility in this direction. The exponents of art may appear in another generation or two to render a service that is not even suggested to-day. There should be a rapidly decreasing

tendency to discuss these boundaries under the rules applied in foreign lands several centuries ago. The writer is of the opinion that riparian boundaries are not to be settled under the common law as it stands, any more than one should anticipate the cure of a case of rheumatism under the same law. They display a decided disposition to become unruly in the hands of exponents of the common law in cases of reliction and accretion.

A decision of one State Supreme Court in a case involving reliction applies the following rules for extending boundary lines between freeholders: First, that boundary lines formerly terminating at the low-water mark be continued at right angles to the shore line (or the contour of the ground surface) as the water recedes; second, that at any time during the process of reliction each freeholder about the lake or swamp shall retain the same proportionate length of shore line he had before reliction commenced. These rules might operate where the basin of the lake or swamp has the form of a right cone. There are no such lakes or swamps.

The tendency of Court decisions is toward a complete recognition of public interest in waters, and, consequently, in the beds of rivers and lakes. A recent decision of the Supreme Court of Michigan holds that a fisherman who follows a stream is not a trespasser.

Legal authorities are likely to see only one clause of the Constitution when dealing with this subject. That much abused Fourteenth Amendment stands out like a beacon when the question of riparian boundaries and riparian lands is under discussion. The Constitution is not altogether silent regarding the public welfare. Only a few freeholders can make a beneficial use of riparian lands. Where beneficial uses may be made of lands adjacent to streams or lakes or under the water thereof, the public could easily grant permits therefor. This would make boundary lines definite and the surveyor could locate them without much difficulty. Such a public policy would also completely destroy the so-called riparian rights claimed frequently by freeholders who are now legalized "dogs in the manger". Every legitimate development is fostered under an intelligent administration of waters. No efficient administration of waters is possible under doctrines framed to support a feudal society.

In theory at least, riparian boundaries are lines marked on the ground to separate waters and lands. Nature furnishes guides which may be safely followed by the surveyor. Aside from navigation interests, rivers and lakes have public aspects which are distinct from any claims or rights which might follow or accompany land tenure. If lakes and rivers are considered only as a part of the natural drainage system, they must be acknowledged to belong to the public. What man does in the field of drainage merely supplements natural systems. If rivers and lakes are to be considered as an important part of public drainage works, engineers must take the next step and state how far freeholders may encroach on their banks and shores without interfering with the free flow of water. The question of boundaries at once arises. If freeholders are permitted to extend their boundaries to the "thread" of streams and to the "middle" of lakes, the public is handicapped when it plans improvements of river channels and lake basins.

Claimants to riparian lands seldom boast of the value of such lands within the hearing of local assessors. If some development, such as water power, is proposed, possibly of great prospective value and importance to the local public, these lands become extremely valuable in the eyes of "riparian proprietors". In most States a power company cannot safely proceed with development work until it purchases large areas of riparian lands. It should only be required to purchase lands it is to flood or occupy. The cost of riparian lands must be added to the construction or development charges, and the local public ultimately pays for them—a penalty for carelessness which permits freeholders to secure title to property that is by nature public.

At present, the surveyor stands at ease, waiting for orders that he can carry into effect. Riparian boundaries should cease to be a subject for academic discussion. The surveyor, being a scientist, hopes that the Courts may soon find some fundamental principles which will encourage growth and give rise to specifications which can be followed in the field. The early authority in geodesy could not serve his generation effectively as long as society insisted that the earth is flat. The surveyor of to-day cannot locate boundaries on the ground where they are referred to such imaginary things as threads of streams and centers of lakes.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

AUTOMOBILE HAZARD IN CITIES AND ITS REDUCTION

Discussion*

By Messrs. Lee S. Trainor, Sidney J. Williams, and A. B. Barber.

Lee S. Trainor,† Assoc. M. Am. Soc. C. E. (by letter).‡—This paper is most valuable and presents clearly the existing automobile hazards in cities and the major points of traffic control affecting these hazards. The writer wishes to call attention to a few factors of particular importance to municipal engineers.

In a recent paper before the Society of Automotive Engineers, T. R. Agg, M. Am. Soc. C. E.,§ emphasized the fact that whereas vehicles were formerly designed to fit the road, to-day pavements are built to suit the vehicle. Several points in the relationship of pavements to the vehicles operating on them, should be given consideration.

The type of street pavement and its condition has an important bearing on the safety with which traffic moves. A street can hardly be considered as giving efficient transportation service unless the pavement on the street is in such a condition, from curb to curb, that vehicles are given the uninterrupted use of the entire width of the thoroughfare. A poor street pavement, the surface of which is irregular, or spotted with holes, offers a hazard to passing vehicles which is often difficult to surmount. Bad holes or rough places cause vehicles to deflect from their line of travel into another line, creating a hazard for oncoming vehicles. It is important, therefore, that heavily traveled thoroughfares be provided with pavements that retain smoothness of surface and that can be maintained with minimum expense and the

* Discussion of the paper by William J. Cox, Jun. Am. Soc. C. E., continued from September, 1927, *Proceedings*.

† Mgr., Highways and Municipal Bureau, Portland Cement Association, Chicago, Ill.

‡ Received by the Secretary, August 8, 1927.

§ "The Design and Construction of Highway Systems," *Journal*, Soc. of Automotive Engrs., Vol. XXI (August, 1927), p. 200.

least interference to the normal flow of traffic. The Keystone Automobile Club, of Philadelphia, Pa., reports that the City of Philadelphia expends $500 000 per year in damages for accidents caused by defective pavements.

Congestion of traffic in large centers of population is mounting more rapidly each year. Unfortunately, the majority of the larger cities in the United States were laid out and developed long before traffic congestion had become a problem. The unprecedented growth of the motor industry and its allied industries, which have produced the present-day volume of traffic, could not have been imagined at that time. Consequently, the street systems of the larger cities have become antiquated and do not render full transportation service.

Because of the lack of proper street width, congestion presents a traffic hazard that is most difficult to overcome. American cities have grown at such a rapid rate that street widening in densely populated areas has become an expensive operation. Congestion is truly a sufficient cause for providing main arteries of traffic with such pavements that the maximum service of the existing width can be obtained. Many American cities are spending millions of dollars annually for street widening in order to reduce the hazard of congestion.

Blind corners, short-radius street intersections, and poor visibility, especially at night, are other hazards encountered by traffic in metropolitan areas. Even the color of a pavement affects the safety of moving vehicles, especially at night. A dark pavement has a tendency to absorb the rays of headlights to such an extent that the limits of the thoroughfare are but dimly defined. It is also more difficult to see pedestrians and parked vehicles at a safe distance. The same is true concerning the amount of light perceived from vehicles approaching from the right or left at street intersections. A light-colored pavement reflects the rays of the head lamps, thus outlining the course for a greater distance ahead. A light-colored pavement also creates enough contrast to give a clear outline of parked vehicles or other hazards that may be placed at the curb. During wet weather, such a pavement is especially conducive to safety because of the contrast in color with surrounding objects.

Heavily traveled arteries should be provided with pavements that are reliable under all conditions. Skidding and sliding with locked wheels are the causes of many accidents on congested thoroughfares. Safety on the boulevard has become more than a matter of good tires and good brakes.

The characteristics of pavement surfaces must be considered. Any pavement that becomes slippery when it is wet is a source of danger often beyond the control of the driver. A rough and bumpy street often induces skidding as tires lose contact with the surface.

Starting and stopping must be accomplished rapidly, and the pavement must do its part perfectly if the hazards are to be reduced. The surface should be such that it will not be slippery and cause skidding on sudden application of the brakes. It should be sufficiently gritty to provide a grip

for the tires, and bring the vehicle to a stop as quickly as possible. A pavement surface of this character is especially beneficial during wet weather. On steep grades it provides enough traction to make ascent easy and safe, and on descent provides sufficient tractive resistance to prevent skidding or side sliding. Such a surface automatically provides hazard elimination for the motorist. The construction of streets and pavements suited to present-day traffic is of material assistance in reducing automobile hazard in cities.

SIDNEY J. WILLIAMS,* M. AM. SOC. C. E. (by letter).†—This paper by Mr. Cox is the first real attempt to develop a scientific formula for the occurrence of automobile accidents in cities. Like any other pioneer job, this method is doubtless open to refinement at the hands of the author or some one else. Further information may correct some of the assumptions that Mr. Cox was compelled to make; but he has at least given all students of the subject something to "shoot at", and his careful and intelligent analysis constitutes a real contribution to the safety movement in the United States.

It would be very desirable to check the formula against data later than 1922 and to check the application of it to the changing experience of one or more cities from year to year.

Many things have happened since 1922 to affect accident causation and accident prevention on the highways. In various cities, there has been opportunity to observe the effect, on the accident record, of various changes in the traffic control situation and in the state of public opinion. It has been found that, in fact as well as in theory, the accident record of a city can be vitally affected by factors other than those stated in the basic formula developed by Mr. Cox; for example, by better traffic regulation and by the right kind of public education. Cities are beginning to accumulate statistical evidence, covering a period of years, to show that continuous and properly organized educational campaigns can be depended on to reduce the accidents in a community just as they do in a factory.

Aside from definite safety educational campaigns, there is the element of the general public attitude toward law enforcement. As a native of the State of Wisconsin, the writer feels able to say with some assurance that the attitude toward law observance in that State and in the City of Milwaukee is very different, and much better, than in the State of Illinois and the City of Chicago, for example. This is probably one reason for the accident record in Milwaukee being better than anticipated, as shown in Mr. Cox's paper. In fact, the writer feels that the general public attitude toward law observance and accident prevention will be found to have an even greater effect on the accident experience of the community, than the street mileage or any other engineering factor. The statisticians and engineers of the National Safety Council, for example, are just beginning to obtain data on which to make quantitative studies of these complicated and important factors. These remarks are in extension and not in criticism of Mr. Cox's excellent paper.

* Director, Public Safety Div., National Safety Council, Chicago, Ill.

† Received by the Secretary, August 13, 1927.

A. B. BARBER,* M. AM. SOC. C. E. (by letter).†—This paper is a distinctly original contribution to the study of the automobile accident and traffic problem and is of special value because of its method of approach. It establishes, in mathematical form, facts that have heretofore generally been supported only by opinion. It is teeming with suggestion, both in and between the lines.

No one who has seriously studied the traffic problem will be surprised at the deduction that, H, the hazard from the operation of the individual automobile, varies directly with population, P. Unfortunately, motor traffic is still generally so poorly organized and pedestrians still roam so freely over the street surface that the assumption of uniform distribution of both these elements is not a material departure from the fact.

The expression, $H \propto \frac{P}{M}$, suggests that H can be lowered by reducing P, or by increasing M. To reduce P signifies relieving a portion of the population (including those in cars as well as those on foot) from the hazard of individually threading their way among moving automobiles. This can be done in a variety of ways. The policeman at the corner with his stop-and-go signal; the school-boy patrol holding up traffic to let the children pass; the automatic traffic signal system; all these are means for decreasing the aggregate amount of the population subject to the hazard indicated in Mr. Cox's initial assumption. That they are successful is shown by the practical elimination of accidents on streets where traffic control is well established and enforced, by officers or by automatic signals. This, as Mr. Cox points out, is one form of traffic "segregation."

Pedestrian control also reduces the amount of the population, P, exposed to the automobile danger. Control, however, is objected to by some pedestrians as a violation of "inalienable rights", just as some motorists regard licensing of drivers as violation of their rights. Both are safety measures, the importance of which is just beginning to be realized.

Under Mr. Cox's formula, H may also be reduced by increasing M. While this holds for the purpose of this paper it may not be true as a permanent measure of accident reduction, for it is natural to suppose that any great increase of street mileage through decentralization or a general spreading out of the population would lead to such an increase of distance traveled per automobile as to require that new element to be taken into account. Proper city planning and zoning measures, however, would serve to offset this tendency. The modern conception of a city or metropolitan region contemplates the location of commercial, industrial, and other centers at appropriate widely distributed points, with housing and community facilites for employees in close proximity to each center. The working out of this conception should economize, not only time and expense of travel, but should also tend to reduce the accident hazard.

* Mgr., Transportation and Communications Dept., Chamber of Commerce of the United States and Director, National Conference on Street and Highway Safety, Washington, D. C.

† Received by the Secretary, August 22, 1927.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

WATER SUPPLY FOR ARMY RAILWAYS IN FRANCE

Discussion*

By Paul M. La Bach, M. Am. Soc. C. E.†

Paul M. La Bach,‡ M. Am. Soc. C. E. (by letter).§—Judging from his discussion, Mr. Atwood|| seems to have gained the impression that the writer wishes to change the entire structure of Army organization. The real purpose of the paper was to give a short account of successes and failures under a given set of circumstances. The suggestion of changes in organization was not directed at any other activities than those in his own department.

The Construction Division was decentralized, that is, the Section Engineers were in charge of construction in a given area. This was a territorial sub-division and each sub-division was independent of its neighbor. The water supply for Army railways was under central control as it could not be handled otherwise. It covered a small area in width, but extended from the Base Ports to the Rhine. In this area, construction, maintenance, and operation were being handled at the same time without a clear line of demarcation between them. To use a railway term, they were "interlocked". To open one line, another had to be closed. The writer's proposal is to put such an area in the same basis as a "Section". This would not change any fundamentals of organization in any manner, but would bring correlated and interlocked activities under one head. This would be similar to actual civil practice so successful for many years.

Armies are organized for combat service and the attempt is usually made to adapt the same organization to constructive activities. Its very virtues in combat sometimes are hindrances under other circumstances. The question of rank and the authority which goes with it have some bearing. When a

* Discussion of the paper by Paul M. La Bach, M. Am. Soc. C. E., continued from August, 1927, *Proceedings*.

† Author's closure.

‡ Engr., Water Service, Rock Island Lines, Chicago, Ill.

§ Received by the Secretary, August 13, 1927.

|| *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1293 *et seq.*

new military unit is invented, or borrowed from civil life, it is usual to put somebody, perhaps an expert, in a staff position to take charge of its activities. If it is a large staff, and he desires to get anything done, he must devote much of his energy to getting priorities. Having had many years' experience the chief of staff will naturally lean toward and favor activities with which he has had personal knowledge. New schemes from civil life, while old subjects to the proponent, are new to him, and so the new staff officer starts with a handicap. In the American Expeditionary Forces, these debates were called the "Battles of Chaumont, Paris, Tours, Toul", etc.

For this the writer would wish to substitute direct action in the case under discussion. Make the responsibility for results and the authority necessary to secure them co-extensive. It was practically impossible to hold meetings with a scattered organization except at wide intervals. To convince those present of the importance of the work under debate was not always possible. Everybody told a similar tale of woe to the Section Engineers.

As an illustration of the workings of the system the writer recalls two communications of the same day about three weeks before the Armistice. One stated that, because of the failure of a small French water station and the fact that the new station had not been completed, 5 miles of trains loaded with ammunition and other supplies were unable to proceed. The other was from a Section Engineer and contained an argument against doing any work on the reinforcement of a water station because the work would never be needed. The benefit of the experience of one was lost in so far as the other was concerned, although both water stations were integral parts of the same machine.

Observation in two wars and several revolutions makes the writer somewhat skeptical on the subject of staff jobs as a cure for troubles. In the World War practically every peace-time staff had to be re-organized after hostilities had begun. The French General Staff, while it had wide control of the railways, did not interfere with actual operation, which was conducted by the railway organization from start to finish. These organizations could not expand to help the American Army as they were already short of both men and material. Most of the personnel had had Army service before the war. When it came to a critical situation at one time it was found that results on the Paris-Lyons-Mediterranean Railway could be obtained more quickly through its organization and the use of German war prisoners than through any military channels. This was due to the centralized organization in direct contact with daily operations.

Decentralized organizations were much more useful in former times than at present. They grew up out of necessity due to slow methods of communication. At present, with the telegraph, telephone, radio, and aeroplane, the same reason does not exist.

In water supply for military railways, the writer's plea, in railroad parlance, is for grade separation as distinguished from interlocking.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

PROBABILITY OF FLOOD FLOWS

Discussion*

By C. S. Jarvis, M. Am. Soc. C. E.

C. S. Jarvis,† M. Am. Soc. C. E. (by letter).‡—The practical value of mathematical analysis applied to flood flow probability is variously appraised among investigators. Not infrequently sweeping denunciations are heard, prompted by cumbersome methods, questionable assumptions, and vague conclusions which are associated generally with involved or intricate processes. On the other hand there are hopeful and earnest attempts at estimating periodicity during 50 000 years, based on very meager records covering 50 years or less; a process comparable with primary triangulation using a magnetic compass in the midst of local attractions.

The writer does not desire to discourage research along the lines suggested by the author, but wishes to emphasize the limitations with which investigators are confronted. With a view to co-ordinating the fragmentary data obtainable from all reliable sources, and weaving them into a connected fabric in which the semblance of a pattern may be traced, if discernible, he has adopted only the elementary, direct methods, and has aimed to preserve the original observed data as it is found. The gratifying results thus far seem to justify a description of the process as applied to flood periodicity.

By way of illustration and comparison, the 57 years of record during the past 61 years for the Tennessee River at Florence, Ala., are arranged in Table 7.

Engineers are interested primarily in either flood volumes or flood heights and their frequencies; therefore, it seems relatively unimportant to include the low-water record except as a background for the peak flows.

Platting on multi-phase logarithmic paper seems to provide the best perspective. Gauge heights may be shown just as they appear in the original

* Discussion of the paper by F. G. Switzer, Assoc. M. Am. Soc. C. E., continued from September, 1927, *Proceedings*.

† Care, U. S. Bureau of Public Roads, Washington, D. C.

‡ Received by the Secretary, August 19, 1927.

record; but it seems preferable to calculate only excess height above what is somewhat arbitrarily called the "flood stage," or danger line. This has the effect of accentuating the curvature and portraying the trend more clearly; it is analogous to placing specimens under the microscope for examination.

TABLE 7.—Flood Records, Tennessee River at Florence, Alabama.

Maximum gauge height, in feet.	Number of occurrences.	Accumulated total.	Average interval, in years, between floods of equal or greater magnitude.
32 or more	1	1	57
31 to 32	1	2	28.50
30 to 31	0	2	28.50
29 to 30	2	4	14.25
28 to 29	1	5	11.40
27 to 28	0	5	11.40
26 to 27	2	7	8.14
25 to 26	2	9	6.33
24 to 25	3	12	4.75
23 to 24	4	16	3.56
22 to 23	4	20	2.85
21 to 22	4	24	2.37
20 to 21	3	27	2.11
19 to 20	8	35	1.63
18 to 19	5	40	1.42
Under 18	17	57	......
Total......................	57	....	......

In Fig. 6 curves for the Tennessee River at Florence are shown, based, respectively, on observed gauge readings and on heights above "flood stage." They illustrate the relative merits and disadvantages as well as the concordance of results. Inasmuch as the maximum reading of 32.2 ft. in a period of 57 years of record is directly on the prolongation of the smooth curve through the other platted points, the normal interval for its recurrence seems to be 57 years, in contrast with the 500-year interval derived by the author and shown in Table 5.* Furthermore, a gauge height of 33.4 ft., indicating a flood discharge rate of 630 000 sec-ft., corresponds to the time interval of nearly 200 years in contrast with the 3 000-year period shown in Fig. 4(*a*).†

The annual flood gauge records for the Mississippi River, at Natchez, Miss., extending at intervals from 1770 to 1801, inclusive, and continuously thereafter to date, reduce to the lowest curve platted in Fig. 6, and indicate that the 1927 flood height may be exceeded by at least 1 ft. during the century. However, the tendency toward higher gauge readings has been much more pronounced since 1870, due in part to channel restrictions and levee extensions, soil erosion, and sedimentation. Influences offsetting that tendency to some degree are the effects of judicious channel rectification and deepening, bank revetment, and stabilized levees for ordinary flood stages, all of which improve the hydraulic elements governing discharge as long as control is maintained.

* *Proceedings*, Am. Soc. C. E., April, 1927, Papers and Discussions, p. 567.
† *Loc. cit.*, p. 568.

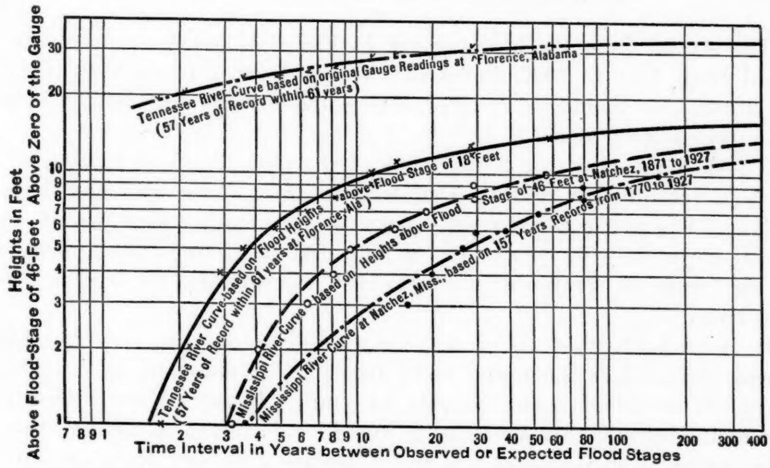


FIG. 6.—FLOOD PROBABILITY CURVES FOR THE MISSISSIPPI AND TENNESSEE RIVERS.

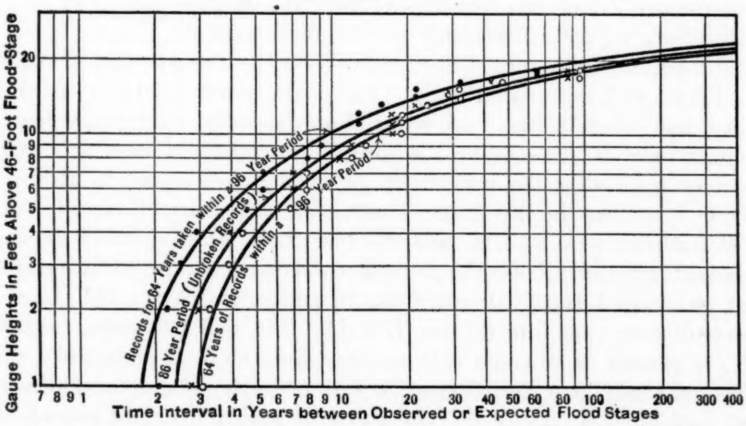


FIG. 7.—FLOOD PROBABILITY CURVES FOR THE OHIO RIVER AT THE UPPER STATION, LOUISVILLE, KENTUCKY.

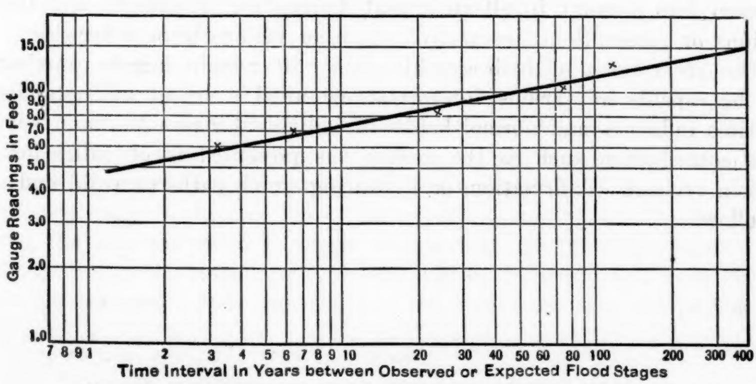


FIG. 8.—FLOOD PROBABILITY CURVES FOR THE NEWA RIVER AT LENINGRAD, RUSSIA.

The second from the lowest curve in Fig. 6 is based on the continuous record from 1871 to 1927, inclusive. It lies from 1 ft. to 3 ft. above the lowest curve, as should be expected, due to the changes and channel infringements wrought by man.

Fig. 7 summarizes the flood stages of the Ohio River at Louisville, Ky., based on intermittent records from 1832 to 1872, and continuous thereafter to date. The upper curve results from the assumption that the 64 annual records cover only 64 years; the middle and the lower curves were based on the same observations, but regarded as covering 86 and 96 years, respectively.

It is probable that the intermittent observations preserve the highest records and neglect the lower ones; therefore, it does not seem logical to regard the period of record as only 64 years. If any of the higher crests were omitted from 1832 to 1872, it would be incorrect to assume that the entire period of 96 years was adequately represented. No doubt the intermediate curve, based on an 86-year record as assumed, is the most reliable. It is noteworthy that the variations from the middle curve are not wide, and that they reduce considerably in the upper ranges.

Fig. 8 portrays the annual flood records during the past 223 years, on the Newa River, at Leningrad, Russia. There, the worst combinations of wind, tide, ice, and rainfall, that may occur simultaneously, have jeopardized most of the city until drastic measures for protection and relief seem to be required. The curve indicating the periodicity of flood stages seems to reduce practically to a tangent in this case. During the 223 years of record, the three highest readings were 13.5 ft. in 1824, 12.4 ft. in 1924, and 10.6 ft. in 1777. The height attained above the normal exceeded 8 ft. in 13 instances, and was at or above 7 ft., 35 times; 6 ft., 70 times; and 5 ft., 163 times. The corresponding average intervals are 17.1, 6.37, 3.19, and 1.36 years, respectively, which are platted as observed data to determine the flood probability curve.

The need has long been apparent for a simple, elementary, direct, and logical method of depicting the trend toward periodicity, and capable of use for extrapolations outside the period of record. The time of recurrence is taken into account in all structural designs for waterways, and the conclusions or assumptions are arrived at either rationally or otherwise.

The great mass of hydrographic data will remain largely unrelated or will be capable of various interpretations until a broad understanding of the time influence and normal habits of individual rivers has been achieved. Each contribution such as the author has presented is of value for comparison, revision, confirmation, or indicating which paths to avoid and which to follow.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE EYE-BAR CABLE SUSPENSION BRIDGE AT FLORIANOPOLIS, BRAZIL

Discussion*

By Messrs. Frederick C. Carstarphen and A. deH. Hoadley.

Frederick C. Carstarphen,† M. Am. Soc. C. E. (by letter).‡—This paper is a worthy contribution to engineering literature. It is complete. It gives the primary premise, the analysis, and the conclusions drawn from the structure in place. Engineers who are interested in suspension bridges, are glad to know that, in design, another milestone has been passed, and they feel that the authors should be complimented on their boldness in using the mid-portion of the cable for the upper chord of the stiffening truss, thus securing in practice, an economy of material that others have contemplated but have not attained.

It is possible that engineers have hesitated to adopt an obvious economy because extreme accuracy must be secured in design and fabrication to attain a satisfactory deck position without excessive use of shims beneath the girders. One-half of the Florianopolis Bridge is without suspender adjustment. The completed structure is a testimonial to the accuracy of the calculations, the care of fabrication, and the competency of erection.

This bridge is noteworthy for the use of eye-bars under tensions of 50 000 lb. per sq. in. Those who have been interested in heat-treated and alloy steels knew that their use under high tensile stresses could not be long deferred. The supremacy of wire cables for suspension bridges having a span of 2 000 ft. and less is now challenged by high-tension eye-bars. The use of overhead or messenger cable in the erection of the eye-bar spans is also a forward step.

From the data presented it might be inferred that the calculation of the initial position of the messenger cable can best be made by trial methods, but such is not the case. It is important to the contractor that the problem be

* Discussion of the paper by D. B. Steinman and William G. Grove, Members, Am. Soc. C. E., continued from September, 1927, *Proceedings*.

† Cons. Engr., Denver, Colo.

‡ Received by the Secretary, July 18, 1927.

correctly solved, so that the eye-bars may be joined and the chain-blocks released; although the latter could be accomplished (with additional expense), by the aid of auxiliary tackle.

The writer has been interested in this problem ever since anchored spans for the track cables of aerial tramways became the vogue. The problem is to determine the position of the empty cable so that, when it is loaded, the resulting tension will not exceed a predetermined value. The cable positions recorded during the erection of the Florianopolis Bridge will serve as an excellent control of the method of calculation.

If a wire cable did not stretch or possess elasticity, an increase in symmetrical loading would not cause it to change its position, but it would increase the tension. The supports are assumed to be rigid. Unfortunately, for simplicity of analysis, this is not the case.

When they are first placed in tension, cables become longer because of the compression of the core and other adjustments of the component wires. This set is noted in all wire ropes, but its magnitude differs somewhat with the use of hemp or wire centers, or lock-coil construction. Reference is here made to the accompanying stress-strain graphs (Fig. 41). They show the effect of progressive and retrogressive loadings applied successively to a $2\frac{1}{2}$-in., 6 by 37 monitor plow steel cable.

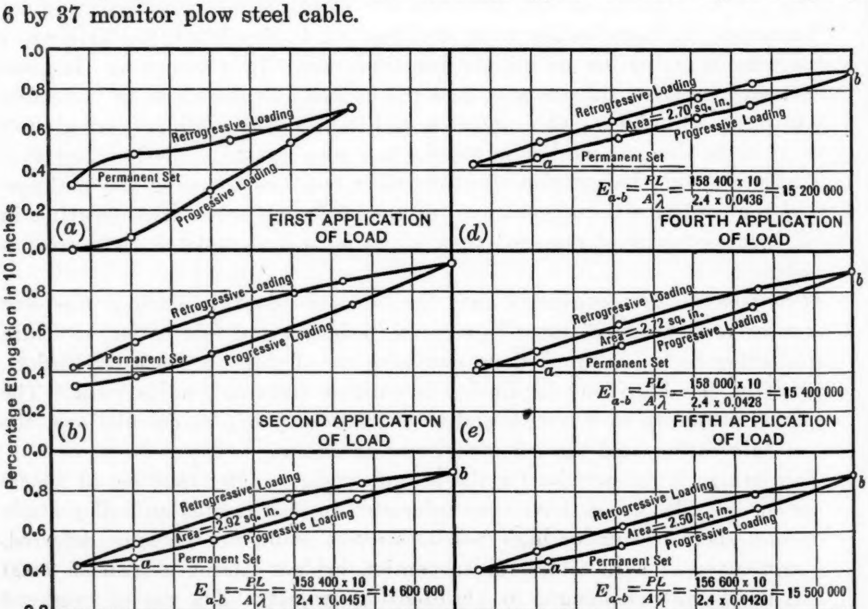


FIG. 41.—RESULTS OF CABLE TESTS.

The permanent set of the rope was not fully developed until after the third application of the load. It will be noted that the graphs do not coincide, but after the third alternation they form a closed curve. They are foot-pound

diagrams, similar to those secured with an indicator, and may be used to determine the magnitude of the internal friction, or resistance to stretching. They might be called frictional hysteresis graphs.

Up to a certain point the modulus of elasticity varies with the tension. As the loads increase the elongations become proportional to the stress, and the value of the elastic modulus is nearly constant. This is so when the internal friction is sufficient to cause the rope to behave as a homogeneous bar, although the value of the modulus of elasticity of the rope is never equal, numerically, to that of the steel from which it is made.

Two applications of a load are not sufficient to determine the true value of the modulus of elasticity (See Fig. 41 (*a*) and (*b*)). To this extent the values given by the authors for 1-in. ropes are open for scrutiny. It will be shown from the behavior of the ropes in service that the modulus of elasticity was nearer 12 000 000 than the 8 300 000 lb. per sq. in. reported to have been used in the calculations. If Fig. 19* is a correct plotting of the stress-strain curve of Specimen *M*, under successive loading, then the crossing of these graphs is difficult to understand. It implies a recapture of the permanent set of the rope, which is not in accord with experience, and leaves the accuracy of the test in doubt.

To secure a factor of safety of 3, ropes having an ultimate strength of 90 000 lb. (monitor plow grade) were selected to sustain the eye-bar chains. The increase in tension per rope from 27 500 lb. in the horizontal portion of the main span to 32 000 lb. in the back-stays is discussed, but the paper seems to be silent as to the additional stress in the cables due to bending them around the tower and anchorage shoes. Was the factor of safety in the ropes approximately 3, or was it less than 2?

A few years ago the inquiry would have proceeded along these lines: What is the direct stress? 32 000 lb. What is the minimum radius of curvature of the shoes? $7\frac{1}{2}$ in. What is the resulting bending stress?† 17 920 lb. What is the total stress? 49 920 lb. If the ultimate strength of the rope is 90 000 lb., what is the factor of safety? 1.8. By the Hewitt and Rankine formulas the factor of safety vanishes. However, these views concerning the magnitude of bending stress in wire ropes could not be reconciled with the experience of the rigger who knows that the ropes do not fall apart when bent around the small sheaves of the usual rope-block, but on the contrary lift loads weighing tons. When a $\frac{3}{4}$-in. 6 by 19 plow-steel cable is bent around a sheave having a 3-in. radius, its ultimate strength is reduced only 3 000 lb.

The reduction in the strength of 1-in. ropes due to bending them around a $7\frac{1}{2}$-in. radius would be about 600 lb. so that this factor need not be considered further.

Wrapping the rope $1\frac{1}{2}$ times around the anchor post prevented the equalizing of tensions in the two parts of the bight, but relieved the rope clips of the greater part of their load. The method of using a nest of sheaves for the anchorage results in low tensions when the ropes are looped around the sheaves and clipped. The present method is a departure from the usual one.

* *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 756.

† See "American Wire Rope," p. 38, Edition 1913.

Since the eye-bar chain does not hang as a parabola, due to the variation in the loading, a determination of its position is in order, so that its departure from this curve may be known. (See Table 13.) The case is illustrated in Fig. 22 (*a*),* the eye-bars carrying only their own weight, the span, 1 115.55 ft., the center deflection, 116 ft., and the loading as given in Table 10.†

The panel length (41.25 ft.) is based on a span of 1 113.75 ft. and must be adjusted to the new length of 1 115.55 ft. The same purpose will be served if the 1.80-ft. difference is divided (0.07 ft. in each of 26 panels) so that the new spacing is 41.32 ft.

The reaction at each support is one-half the sum of the span loads, or 213 120 lb. With a center deflection of 116 ft., the span is in equilibrium when the horizontal component of the tension amounts to 519 527.8 lb. To find the deflections of a parabolic span having the same tension and center deflection, the value of the panel-point loads must be decided. The natural tendency would be to average the tabulated values to 16 393 lb., but if these loads are used with a 116-ft. center deflection, the tension will be more than that developed by the actual loading. The equal loading may be easily found by the formula:‡

$$y = \frac{g\,s\,n}{4\,t}\left(1 - \frac{n}{2}\,q\right) + \frac{w\,s^2}{8\,t} \pm \frac{h}{2} \quad \text{.................. (1)}$$

in which,

y = center deflection.
g = load.
n = number of loads.
t = horizontal tension.
q = ratio, load spacing to span.
s = span.
h = difference in elevation between the supports.

Solving:

$$116 = \frac{g \times 26 \times 1\,115.55}{4 \times 519\,527.8}\left(1 - 13\,\frac{41.32}{1\,115.55}\right)$$

$$g = \frac{116}{0.0072345} = 16\,034.3 \text{ lb.}$$

Notice that the last two terms drop out of this equation because n and h are zero.

Having discovered the loading, the deflections can now be found most easily by the sum of the first and second differences.§ These differences are numerically equal and are found from the expression:

$$-\frac{a\,g}{t} = -\frac{41.32 \times 16\,034.3}{519\,527.8} = 1.27527.$$

The deflections of a series of equal loads on a parabolic arc as compared with those at Florianopolis, both with a tension of 519 527.8 lb., are as given in Table 13.

* *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 757.
† *Loc. cit.*, p. 750.
‡ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1388, Case III.
§ *Loc. cit.*, p. 1388.

TABLE 13.—COMPARISON OF DEFLECTIONS IN FLORIANOPOLIS CABLE AND TRUE PARABOLA.

Panel points.	0.	2.	4.	6.	8.	10.	12.	14.	16.	18.	20.	22.	24.	26.
Parabolic curve deflections, in feet.	0.00	16.53	31.83	45.86	58.61	70.09	80.29	89.22	96.87	103.25	108.35	112.74	114.73	116.00
Florianopolis deflections, in feet..........	0.00	16.95	32.48	46.64	59.44	70.91	81.03	89.86	97.41	103.68	108.65	112.34	114.78	116.00

The differences in these deflections illustrate the influence of small changes in loads on the position of the equilibrium polygon. With a center deflection of 115 ft., the tension becomes,

$$\frac{116}{115} \times 519\,527.8 = 524\,045.1 \text{ lb.}$$

This is the horizontal component of the tension required to support the eye-bars with a center deflection of 115 ft. The number of ropes required to support the chain and their own weight may next be approximated. One-inch monitor plow, 6 by 19 hoisting ropes have a tabulated breaking strength of 90 000 lb. With a factor safety of 3, the working tension can be 30 000 lb. The useful load tension is three-fourths of the working tension, or 22 500 lb.; $\frac{524\,045.1}{22\,500} = 25$ ropes, thus checking nearly the 24 ropes used. The actual tensions are less than 30 000 lb. so that checks the use of 24 ropes.

The manner of erection now must be considered. If the eye-bar chain is joined in the center of the span, the messenger cables must support all loads as well as a 20 000-lb. traveler, causing an increase in the tension of the cables that can not be ignored. This method will be the easier for the erection crew because it insures the proper position of the eye-bars along the span. However, if the eye-bars are started at the center of the span, the tower connections can be made without the traveler being at the center. The contractor is to be complimented on the successful manner in which the eye-bars were handled, using the last method.

The panel loading of the messenger cables may be taken as 16 034.3 + 510 (chain-blocks) + 106 lb. for clips, or a total of 16 650 lb. The weight per foot of cable is 24 by 1.58 = 37.92 lb. The horizontal component of the tension of the assembly, may be found by the formula * extended to include the cables, thus:

$$t = \frac{g\,s\,n}{4\,y}\left(1 - \frac{n}{2}\,q\right) + \frac{w\,s^2}{8\,y} \qquad (2)$$

$$= \frac{16\,650 \times 1\,115.55 \times 26}{4 \times 115}\left(1 - \frac{13 \times 41.32}{1\,115.55}\right) + \frac{37.92 \times 1\,115.55^2}{8 \times 115}$$

$$= 544\,335.84 + 51\,291.72 = 595\,627.56 \text{ lb.}$$

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1388.

The slope of the tangent at the support is easily found as follows:*

$$\tan \beta = \frac{n\,g}{t}\left(1 - \frac{(n+1)\,a}{2\,s}\right) + \frac{w\,s}{2\,t} \qquad (3)$$

$$= \frac{26 \times 16\,650}{595\,627.56}\left(1 - \frac{13.5 \times 41.32}{1\,115.55}\right) + \frac{37.92 \times 1\,115.55}{2 \times 595\,627.56}$$

$$= 0.36335 + 0.03551 = 0.39886 = \tan 21^\circ\ 45'$$

The tension of the ropes at the tower of the long span is:

$$595\,627.56 \text{ lb.} \times \sec 21^\circ\ 45' = 641\,282.41 \text{ lb.}$$

It will be noted that the horizontal tension of the messenger cables (when supporting the eye-bars at a 115-ft. center deflection, span 1 115.55 ft.) is 595 627.56 lb. instead of 660 000 lb. as mentioned.† The tension of the abutment cable is:

$$595\,627.56 \text{ lb.} \times \sec 30^\circ = 687\,770 \text{ lb.}$$

compared with the 770 000 lb. reported.

These tensions may be reduced to stresses in the single ropes, as: Abutment span, 28 657 lb.; main slope, 26 720 lb.; and horizontal tension, 24 817.8 lb. The factor of safety is 3.14, a figure well within that desired by the contractor.

The length of the loaded span may be approximated by the formula:‡

$$L = s + \frac{8}{3}\,\frac{y^2}{s} \qquad (4)$$

$$= 1\,115.55 + \frac{2.6667 \times 115^2}{1\,115.55}$$

$$= 1\,146.15 \text{ ft.}$$

The approximate nature of this formula, when the deflection exceeds one-twentieth of the span, is well known.

It is easy to ascertain the change in slope at the panel points from the relation of $\frac{g}{t}$ to the load intersection angle, and $\frac{w\,a}{t}$ to the cable angle. The sum of these tangents is $0.03059 = \tan 1^\circ\ 45'$.

The length of the loaded span may be determined for an odd number of links by taking the summation of the hypotenuse of triangles composing the equilibrium polygon of the deflection, as follows:

$$L = a\,[2\,\Sigma \sec \alpha + 1] \qquad (5)$$

$$L = 2 \times 41.32 \times 13.35156 + 41.32 = 1\,144.689 \text{ ft.}$$

Having found the length of the loaded span (Table 14), the next step is to find the length of these cables, or the tension, when hanging as an empty span, so that when carrying the given loads, the tension will not exceed 595 627.56 lb.

For convenience, a single rope will be used in making this calculation; weight, 1.58 lb. per ft.; and tension, when loaded, 24 817.8 lb., say, 24 820 lb.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1387.

† *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 752.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1393.

It is obvious that the loading of the empty span increases its length and increases the tension to the limiting amount, hence the loaded span length, L, is the sum of the length of the empty span and the stretch, and so the expression takes the following form*:

$$L = s + \frac{w^2 s^3}{24 t^2} + \lambda (T - t) s \qquad (6)$$

TABLE 14.—Tabulated Computations for Σ sec α.

Member.	Angle.	Secant α.	Deflection, in feet.
26-26	0	....	115.00
24	1° 45′	1.00009	113.74
22	3° 30′	1.00187	111.21
20	5° 15′	1.00425	107.41
18	6° 00′	1.00551	102.36
16	7° 45′	1.00922	96.04
14	9° 30′	1.01391	88.45
12	11° 15′	1.01960	79.60
10	13° 00′	1.02630	70.29
8	14° 45′	1.03408	58.11
6	16° 30′	1.04295	45.47
4	18° 15′	1.05296	31.56
2	20° 00′	1.06418	16.39
0	21° 45′	1.07664	0
Σ sec. α = ...	....	13.35156	....

The position of the cable, will be ascertained by using both 8 500 000 and 12 000 000 lb. per sq. in. for the value of the modulus of elasticity in tension. The value of the elongation per 1 000 lb. per ft. is:

$$\lambda = \frac{1\ 000 \text{ lb.}}{0.41 \times 8\ 500\ 000} = 0.000287 \text{ ft.}$$

$$\lambda = \frac{1\ 000}{0.41 \times 12\ 000\ 000} = 0.000203 \text{ ft.}$$

Substituting in Equation (6):

$$1\ 144.69 = 1\ 115.55 + \frac{1.58^2 \times 1\ 115.55^3}{24\ t^2} + 0.000287\ (24\ 820 - t)\ 1.11555$$

Reducing:

$$21.19 = \frac{144\ 400\ 000}{t^2} - 0.00032\ t$$

This is a cubic equation, having one real root and is in a form that admits of easy solution by means of a slide rule, thus:

$b = \frac{t}{1\ 000}$	$\frac{144.4}{b^2}$	$-\ 0.32\ b$	21.19
2.56	22.05	0.82	21.23
2.57	21.90	0.82	21.08
2.565	22.00	0.81	21.19

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1387.

Therefore, 2 565 lb. would be the erection tension if there were no adjoining spans or back-stays. The amount of run of rope from these spans must be found when the tension changes from 2 960 to 28 659 lb. actual, or 25 700 lb. The chord lengths of the spans are 272.23 and 292.16 ft.; sag, 0.26 and 0.27 ft.; stretch, 2.02 and 2.16 ft.; and run, 2.28 and 2.43 ft. The total run is 4.71 ft. Allowing for the increase in length of the temporary eye-bars when under tension, the allowance of 5 ft. for run (made during erection), was correct. The span of the empty cable, therefore, would be increased to 1 115.55 + 5.00 = 1 120.55 ft., and the equation for tension becomes:

$$1\ 144.69 = 1\ 120.55 + \frac{1.58^2 \times 1\ 120.55^3}{24\ t^2} + 0.000287\ (24\ 820 - t)\ 1.12055$$

$$16.16 = \frac{146\ 300\ 000}{t^2} - 0.000287\ t$$

$b = \frac{t}{1\ 000}$	$\frac{146.3}{b^2}$	$-0.3216\ b$	16.16
2.92	17.1	0.94	16.16

Therefore, if the main cables are to have a center deflection of 115 ft. when loaded, and their modulus of elasticity is 8 500 000 lb. per sq. in., the pilot cable should be erected with a horizontal tension of 2 920 lb., or a center deflection of 84.12 ft.

Instead of 84 ft., a deflection of 87 ft. was used, and it will be noted that the loaded cables deflected to 113, and not 115 ft. This loading of the messenger cable may be regarded as a method of determining the modulus of elasticity. The cables, when erected with a deflection of 87 ft., were under a horizontal tension of 2 820 lb. each; at 99 ft., the tension was 2 480 lb. Loaded to a deflection of 113 ft., the tension was 25 476 lb. The difference in the length of the cable between a deflection of 113 ft. and 87 ft. is 7.54 ft.; the difference in tension is 22 656 lb.; from which the modulus of elasticity in tension may be computed:

$$E = \frac{22\ 656 \times 1\ 703}{0.41 \times 7.54} = 12\ 400\ 000 \text{ lb. per sq. in.}$$

It has been stated* that 12 000 000 lb. per sq. in. was adopted for wire ropes with hemp centers, after making numerous and careful tests, and, therefore, it appears that these ropes were not exceptional in this regard.

The tension equation, using elongation based on a modulus of 12 000 000 lb. per sq. in., becomes:

$$1\ 144.69 = 1\ 120.55 + \frac{1.58^2 \times 1\ 120.55^3}{t^2} + 0.000203\ (24\ 820 - t)\ 1.12055$$

$$18.44 = \frac{146\ 300\ 000}{t^2} - 0.000229\ t$$

The value of t is 2 760 lb. and the corresponding deflection is 89 ft. It is known from experience that if these empty cables were so placed they would sag to the desired 115 ft. when loaded. Fortune favored the contractor, for

* "American Wire Rope," p. 33.

the excess of cable length, resulting from approximate methods of calculation offset the effect of using 8 300 000 lb. per sq. in. for the modulus of elasticity.

The messenger system of erecting eye-bar spans has proved to be a meritorious one, and it has been a pleasure to study its application to the Florianopolis Bridge.

A. deH. Hoadley,* Jun. Am. Soc. C. E. (by letter).†—The writer's interest in the Florianopolis Bridge began in 1925 when Charles M. Spofford, M. Am. Soc. C. E., suggested that he make a stress analysis of this bridge for thesis work. The writer presents a brief account of the method used and shows that it is merely another form of expressing the influence line equation given by Mr. Steinman for the method of elastic weights.

Castigliano showed that if the stress in each member of a structure which is indeterminate to the first degree, as is the bridge in question, is expressed in terms of a single unknown internal stress, the derivative of the internal work with respect to the unknown stress will be equal to zero, thus giving an equation from which the unknown can be found. Here the panel load, x, due to a unit load at some point on the bridge, was taken as the unknown. This load was assumed to be equal at all points because the truss deflection was neglected. The stress, U, in each member was then expressed in terms of x, and the unit load and the value of the derivative of the internal work with respect to x, were found and equated to zero. It was decided to neglect the effect of shear, so that the internal work of the truss could be expressed in terms of the bending moment, M, and the moment of inertia of the truss. This gave the equation:

$$\frac{d\,w}{d\,x} = 0 = \sum \frac{U s}{A\,E}\,\frac{d\,U}{d\,x} + \frac{E}{E\,c}\sum \frac{U s}{A\,E}\,\frac{d\,U}{d\,x} + \frac{A}{A_1\,A_2}\sum \int \frac{M}{\delta^2}\,\frac{d\,M}{d\,x}\,d\,y$$

in which,

U = total stress in member.
s = length of member.
A_1 = area of top chord.
A_2 = area of bottom chord.
$A = A_1 + A_2$.
δ^2 = depth of truss at any point.
$$I = \frac{A_1\,A_2\,\delta^2}{A}$$

Solving this equation for a unit load, successively placed at different points on the span, gave the influence line for x from which the H-influence line was readily obtained.

The first term deals with the work in the towers, the second with that in the cable and back-stays, and the third with that in the stiffening truss.

In order to bring the last term of the writer's equation into an integrable form, δ had to be expressed as a function of the horizontal distance, y. For the end thirds, δ varied directly with y. In the middle third it was

* Instr., Union Coll., Schenectady, N. Y.

† Received by the Secretary, August 3, 1927.

found that a parabola through U-16 and U-27 nearly coincided with the cable curve, so that δ was the distance from that parabola to the one on which the lower chord was laid.

The following results were obtained.

Panel Point.	Ordinate to H-Influence line.	Panel Point.	Ordinate to H-Influence line.
2	0.2012	16	1.2831
4	0.3705	18	1.3926
6	0.5445	20	1.4896
8	0.7104	22	1.5773
10	0.8684	24	1.6367
12	1.0108	26	1.6774
14	1.1516		

This shows the elastic weight value to be 5% less than the least work value at Panel Point 26. If the elastic weights for the diagonals are omitted, the writer found, on the basis of the other elastic weights given by Mr. Steinman, the H-influence line ordinate at Panel Point 26 to be 1.5621. This is 7% less than the least work value in which the diagonals were neglected and shows the omission of diagonals to reduce the elastic weight values 2 per cent.

The effect of the top chord members in the middle third of the truss appears in both the second and third members of the least work equation. In the second term there is positive work due to cable tension and in the third term also positive work usually due to compression from bending in the truss. The actual internal work is not the sum of that caused by the forces acting separately (as here taken), but is that due to the algebraic sum of the forces. This may account for a part of the discrepancy between the least work and elastic weight values.

The least work equation readily lends itself to a study of the effect of varying the chord sections on the H-influence line. Once having evaluated the integrals for the different positions of the unit load, it is a simple matter to substitute different values of $\frac{A}{A_1\,A_2}$ for each section and compare the results obtained. The third term of the equation, which is that involving the chord members, has much greater weight than either of the other two terms. Consequently, the factor, $\frac{A}{A_1\,A_2}$, is an important one. This is shown in the summation of $u\,\Delta\,s$ (the denominator of the H-equation) given by Mr. Steinman in Table 5.* Here, $u\,\Delta\,s$ is given as $1\,188.7069 \times 10^{-8}$ ft-lb., to which the chord members alone contribute $1\,051.4886 \times 10^{-8}$ ft-lb.

The influence line equation given by Mr. Steinman can be obtained by assuming H as the unknown, expressing the internal work of each member in terms of H and of the unit load, and equating the derivative of the internal work with respect to H to zero.

* *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, pp. 740–741.

S = length of member.

u = stress in member due to $H = 1$.

Z = stress in member of a simple span truss with same dimensions as the stiffening truss due to unit load.

U = total stress in member $= Z + uH$.

The total internal work is,

$$W = \frac{U^2 S}{2 A E}$$

$$\frac{d W}{d H} = 0 = \sum \frac{U S}{A E} \frac{d U}{d h} = \sum \frac{Z u S}{A E} + \sum \frac{u^2 H S}{A E}$$

$$H = \frac{-\sum \frac{Z u S}{A E}}{\sum \frac{u^2 S}{A E}} = \frac{-\Sigma Z \Delta s}{\Sigma u \Delta s}$$

as

$$\Delta s = \frac{u S}{A E}.$$

If only one or two points on the H-influence line were desired, they could be found more rapidly on the basis of this equation than by the method of elastic weights. However, if all the ordinates were desired, the method of elastic weights will be much shorter, because $\Sigma Z \Delta s$ has to be evaluated for every ordinate to the influence line. Taking the value of $\Sigma u \Delta s$ given in Table 5 and the values for Δs given in Tables 3,* 4,† and 5, the writer computed $\Sigma Z \Delta s$ for a unit load at the center of the bridge and found the corresponding value of H to agree (as it should) with that found by the method of elastic weights.

* *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, pp. 736-737.

† *Loc. cit.*, pp. 738-739.

way
fai
ref
a c
qua

gre
ap
lar
de
th
bl
rig
w

w
ac
n
ti
d
f
b

1

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE RELATION OF HIGHWAY TRANSPORTATION TO THE RAILWAY

Discussion*

BY MESSRS. W. W. CROSBY AND WILLIAM T. LYLE.

W. W. CROSBY,† M. AM. SOC. C. E. (by letter).‡—"The Relation of Highway Transportation to the Railway", as set forth by Mr. Budd, seems to be the fairest statement of fact that has been printed on this subject. It seems refreshingly frank, lacking in prejudice, thoughtful, and even prophetic; with a calmly judicial atmosphere permeating it which adds to its convincing quality.

Perhaps one might wish that brevity had been sacrificed for the sake of greater clarity in some cases. For instance, in his "Inventory",§ it is not apparent just how the "investment" figures are reached. Do they include land or right-of-way values in any or all cases? "Cost" figures always suggest detailed explanation and scrutiny before acceptance. Just what is meant by the statement that "twenty-five years ago the inventory would have been blank so far as modern highway transportation is concerned"? Were not the rights of way existent and valuable? And are not some of the important highways of to-day in the main the same highways of twenty-five years ago?

The essentiality of the railways is well set forth by Mr. Budd, and the writer is glad to see it emphasized contemporaneously with the recognition accorded by the author to the importance and supplemental abilities of the newer means and forms of transportation. In these days of "Consumptionism" there is certainly a need of brakes on the apparently growing tendency to throw away the old for the sake of something new. The arguments for economy in transportation for the sake of public welfare seem soundly, if briefly, stated.

---

* Discussion on the paper by Ralph Budd, M. Am. Soc. C. E., continued from August, 1927, *Proceedings*.

† Cons. Engr., Baltimore, Md.

‡ Received by the Secreta-y, July 1, 1927.

§ *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 793.

Mr. Budd points out correctly that the automobile is not of American origin, but there may be some argument—of the "hen or the egg" variety—with his statements as to the relation of automobiles and highway improvement. From his own observation and experience, the writer would be inclined to put the highway before the automobile.

Arguing most convincingly in the matter of motor-bus competition with, or in addition to, railway transportation, the author reaches conclusions that, with perhaps one exception, appeal strongly. As to the conclusion that, "The public must support whatever transportation agencies are maintained and should not undertake two where one will suffice", he may be correct theoretically, but at the same time give a wrong impression.

Is not a reasonable amount of competition needed in transportation now, as well as in trade, to insure the proper economic "life"? If railway officials were always able to act as calmly and fairly as Mr. Budd has written, there might be less need for competition; but even with the belief that,

"—the days are hastening on
By Prophet-Bards foretold,"

it still seems advisable to allow reasonable competition in many lines of activity, including transportation; not putting into the "one basket" of "railway direction" all the "eggs" of the "new form of transportation"; nor, indeed, in that basket until that railway direction shall have substantially recovered from its past handicaps of "public damnation", Government control, etc. Mr. Budd's last sentence is a mildly expressed, pithy summary of the whole matter.

William T. Lyle,* M. Am. Soc. C. E. (by letter).†—The fundamental economic principle governing the relations between railway and highway transportation should be co-ordination. The public cannot afford superfluous and overlapping service, but requires, and should demand, a smooth-running and efficient transportation system. Without it the national commercial and industrial structure will disintegrate.

The following information, collected by the Interstate Commerce Commission, is based on replies received from 164 out of 176, Class I line-haul steam railways, including all the larger companies; 377 out of 635 Class II and Class III steam railways; and 118 out of 260 unclassified electric railways.‡

Steam Railways.—Buses.—No buses are operated by subsidiaries in terminal service as compared to 506 in line service. Leading Class I railroads have reported subsidiary bus lines operating buses for service in connection with their own operations, as follows: The New York, New Haven and Hartford, 118; the New York Central, 81; and the Union Pacific System, 39; all in line service. In addition, the Great Northern reported 74 in competition with its own operations and 83 in competition with other railroads. The railroads of Class I owned 225 of the buses in operation and those in Class II and Class III, owned 23.

* Prof. of Civ. Eng., Washington and Lee Univ., Lexington, Va.

† Received by the Secretary, July 28, 1927.

‡ Docket No. 18 300.

Motor transport companies and individuals operate 1 511 buses in terminal service to Class I railways and 19 in terminal service to Class II and Class III railways. Only one of these is engaged in interstate traffic. The buses operated by motor transport companies and individuals in line service competing with Class II and Class III railways, number nearly 600.

The New York Central reports 473 buses; the Chicago and Eastern Illinois, 246; the "Big Four", 349; the Pennsylvania, 292; and others, 170; to a total of 1 530 buses operated by motor transportation companies and private individuals in competition with Class I railroads in terminal service. On line service the Pennsylvania reports 2 463 buses competing; the Baltimore and Ohio, 838; Chicago, Milwaukee, and St. Paul, 1 234; Atchison, Topeka, and Santa Fé, 1 079; the Southern Pacific lines 1 891; and other Class I roads, 11 594; making a total of 19 099 competing buses. There are also 1 253 independently operated buses that do not compete, but supplement the service of the steam roads. Of the combined total of 20 352, 2 848 are engaged in interstate traffic, 14 193 in intrastate traffic, and 3 311 in traffic of both kinds.

Steam Railways.—Motor Trucks.—Steam railways of all classes report somewhat similar conditions with respect to motor trucks. Only 27 trucks are operated directly by first, second, and third-class railroads in terminal and line service and only 57 trucks by subsidiaries.

The railroads own 56 of the motor trucks in operation, and the motor transportation companies subsidized by them do not own any. The New York Central declares that 526 trucks are operated by motor transportation companies and private individuals in terminal service along their lines; the Pennsylvania, 603; the "Big Four", 381; and all others, 407; making a total of 1 917 trucks. Of this number only 13 affect the service of Class II and Class III railroads, the remainder being reported by Class I roads.

In line service, the reports are: New York, New Haven and Hartford, 2 229; New York Central, 2 648; Pennsylvania, 6 564; Chicago, Milwaukee, and St. Paul, 1 978; Atchison, Topeka, and Santa Fé, 3 662; the Union Pacific System, 3 531; and all other Class I roads, 22 595; or a total of 43 207 trucks in competitive service. These roads report, likewise, that there are 225 privately owned trucks operating in connection with their own lines.

In line service, Class II and Class III railroads are affected by only 545 private motor trucks, while railroads in Class I have 43 207 competing against them. Of this last number, about one-half are engaged in intrastate traffic and the remainder in interstate traffic, or a combination of both kinds.

A similar analysis of tabulated data obtained from operators of electric railways reveals the following facts.

Electric Railways.—Buses.—There are 401 buses in terminal service, practically all in intrastate traffic. In line service, working within State lines, there are 350 buses; 751 in all. Of this total, 300 are competing with the railroad reporting; 100 are competing with other roads; and 351 are not competing with any railroad.

Subsidiaries operate 190 buses in terminal service, chiefly in intrastate traffic; and 406 in line service of which 59 are engaged in interstate traffic,

178 in intrastate traffic, and 169 in traffic otherwise unclassified. Subdividing further, 339 of the total operate in connection with the electric roads reporting; 226 compete with them; and 31 compete with other railroads.

There are 955 buses owned by electric railway companies from the eight National regions, with the Southern region listing none. There are only 75 buses owned by subsidiaries.

Motor transport companies and individuals operate 75 buses in terminal traffic, all of an intrastate character; and 1 356 in line service, of which 170 are distinctly in interstate service, 586 in intrastate service, and 600 not classified. In the line service, only 1 040 buses compete with electric lines.

Electric Railways.—Motor Trucks.—Railroad companies report that they operate 30 trucks and their subsidiaries 19, both being of negligible significance. They state that they own 36 of the trucks, the subsidiaries owning none.

Motor transport companies and individuals operate 27 trucks in terminal service and 2 394 trucks in line service. Of the 2 394 trucks in line service, 77 operate in interstate service; 1 297 in intrastate service; and 1 020 in service common to both.

Of these same 2 394 trucks, 43 are run in connection with electric railway operations and 2 351 in competition with them. Of the 2 351 trucks, the large number of 1 340 are operated by the Pacific Electric.

Comment.—This investigation shows that a comparatively small percentage of buses and a large but undetermined percentage of motor trucks are operated in interstate traffic.

The information presented reveals a very large degree of competition in transportation service, which should be closely investigated in each individual case. The principle laid down by the author that common carrier motor trucks should not be permitted to operate in competition with railways, except when they are a real public convenience or necessity, appears to be thoroughly sound. His view that the railways must recognize that public convenience and necessity require the development of transportation on the highways, and that they should not attempt to eliminate motor-vehicle competition by arbitrary means, appears to be liberal. Such competition should be subject to proper control. Consideration should be given to the question whether or not highway passenger transportation can be conducted more advantageously under railway direction. The importance of National, State, and joint board regulation is apparent and, for the present, each case should be carefully determined on its own individual merits.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

SOME PHASES OF IRRIGATION FINANCING

Discussion*

By Messrs. R. P. Teele and J. L. Burkholder.

R. P. Teele,† Esq. (by letter).‡—This paper agrees so completely with the writer's views§ that there is little occasion to add anything to the author's discussion. It is needless to take the space to re-state points of agreement, but it would be well to discuss a few other points.

In his opening paragraph,‖ Mr. Henny states that the West is dependent on irrigation for its development and, that "it must grow the bulk of its own food requirements to permit industries to grow and its resources to be developed." The implication is that the West needs more irrigation to provide its own food requirements. Yet, in the arid region, there is not one section that contains a large irrigated area that does not have to go outside to market its principal crops. In traveling over several of the larger projects, it will be found that a conspicuous feature of the local news everywhere is the number of cars of products that have "rolled" to outside markets. The point is that the bulk of the land now irrigated, is not used to produce a local food supply, nor will the bulk of the land reclaimed in the future be used for that purpose.

Mr. Henny argues, as do many others, that additional reclamation should not wait on higher prices and an actual demand for products, because it takes several years to develop a large irrigation project. In referring to the Census figures for 1920 and 1925, he gives an effective rebuttal of his own argument. He gives Census figures showing a decrease of 31 000 000 acres in "improved land" during the period 1920 to 1925. That land went out of use

* Discussion on the paper by D. C. Henny, M. Am. Soc. C. E., continued from September, 1927, *Proceedings*.

† Agricultural Economist, Bureau of Agricultural Economics, U. S. Dept. of Agriculture, Washington, D. C. Mr. Teele died on August 31, 1927.

‡ Received by the Secretary, July 1, 1927.

§ *Bulletin No. 1257*, U. S. Dept. of Agriculture, 1924.

‖ *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 896.

because of low prices. Most of it, and much other land, can and will come into use with higher prices. That 31 000 000 acres will act as a shock absorber and give ample time to build a few new projects before the shortage of tillable land becomes acute, even if the projects are not started until the pressure is felt.

It is difficult, if not impossible to follow Mr. Henny's reasoning in his discussion of the insistent demand for further reclamation in the West, when he states that:

"The demand, moreover, is for new projects on an increasing scale of magnitude and is so general and unanimous that it cannot logically be ascribed to motives reminiscent of 'pork barrel' appropriations."

It will be recalled that Congress recently provided for writing off about $27 000 000 of construction costs on Government projects, and extended the period of repayment of construction charges from 20 to 40 years without interest and that a committee is now in the West hearing arguments for revising existing contracts on the ground that farmers cannot meet their payments.

Mr. Henny supplies the explanation of the "general and unanimous" demand for new projects and the refutation of his own statement:

"The comparative ease with which these and other relief measures were secured from Congress, and the fact that not in a single case, even after the passage of the newer Acts, has the Government enforced collection by foreclosure under lien, and the ready encouragement of farmers' demands by Congressmen, have had an unfortunate effect on the psychological condition * * *."

He takes issue with the writer's arguments against a National subsidy to land reclamation, quoting: "There is no justification for a National subsidy to land reclamation. If local interests justify the subsidizing of land reclamation the subsidy should be local;" and states that he believes it is too sweeping. This belief is justified and the statement should be modified by limiting it to the present time. In a book now in publication* this matter has been discussed more fully. There, it is suggested that the cost be spread in proportion to the benefits. After discussing the lack of correlation between demand for additional reclamation and the need for the products to be grown, the writer states:

"The real problem for the future of reclamation is the devising of a policy that will so strengthen that relation that it will control the rate of expansion. The most effective means of accomplishing this will be to attach financial liability to demand. That is, let those who demand the undertaking of a reclamation project assume an effective liability for the cost. This can be accomplished through the district system already utilized to a considerable extent in irrigation and almost exclusively in drainage reclamation by providing for including in districts not only the land to be actually irrigated or drained, but also the territory to be benefitted indirectly, including cities and towns in the vicinity. The extent of each district should

* "The Economics of Land Reclamation," by R. P. Teele, The A. W. Shaw Co., Chicago, Ill., 1927.

be determined by conditions in its particular case, and, in theory, cost should be apportioned in proportion to benefits, although this is an extremely difficult task in practice.

"The apportionment of cost in proportion to benefits should be a fundamental part of our reclamation policy of the future, if there is to be any public participation in reclamation. This policy can be put into effect * * * by having districts formed under public supervision and including within them all the property that will be benefitted, not overlooking the towns and cities whose chambers of commerce are so active in demanding the undertaking of new enterprises; and requiring the approval of the districts by the counties and States if there are to be county and State contributions; and by Congress if there is to be a Federal contribution.

"Certainly, if reclamation is to be undertaken because of public benefits, the reclaimed land should not be expected to repay the whole cost.

"Such a scheme is susceptible of universal application. If it appears that there will not be such a general benefit as to justify the inclusion of any land outside of that to be reclaimed, the boundaries can be fixed on that basis, and the reclaimed land alone will be liable for the cost; if only lands in the immediate vicinity will be benefitted, the boundaries can be fixed on that basis; if the benefit is county-wide, counties may undertake reclamation; if it is State-wide, the States may undertake it; and if the benefit is interstate, interstate districts may be created."

This shows what the writer meant by local subsidy. Mr. Henny compares land reclamation subsidies to railroad subsidies, rivers and harbors subsidies, and the protective tariff. There is one very important difference that he and others usually overlook. The railroads are public service corporations and furnish service to all who may have occasion to use them; the rivers and harbors and highways are open to all who may pass that way; and the tariff affords protection (if it does) to any one who may produce the protected article; while an irrigation system supplies water to designated lands and to no other lands. The immediate benefit is to those lands, and the owners of those lands have exclusive use of the water supplied. The public has no right whatever to its use.

J. L. BURKHOLDER,* M. AM. SOC. C. E. (by letter).†—This paper is of timely interest to every one having the welfare and development of the Western United States at heart. Under present conditions, and under irrigation district methods, it is almost impossible to finance large irrigation developments. There is an apparent lack of demand for new farms, and it is doubtful if projects, requiring settlement of people on the land, should be recommended for construction at this time. This fact, and the recent criticism of the Federal Reclamation Bureau and the apparent inability of some of its projects to meet construction payments, certainly indicates that irrigation problems should be given thorough study.

The paper has listed the various agencies heretofore utilized for accomplishing irrigation work as follows: (1) Federal; (2) State; (3) Carey Act; and (4) irrigation districts. The last of these agencies is divided into two general groups; (A) districts with land values greater, and (B) districts

* Chf. Engr., Middle Rio Grande Conservancy District, Albuquerque, N. Mex.

† Received by the Secretary, July 23, 1927.

with land values smaller, than the face value of the proposed bond issues The first-mentioned districts have been generally successful.

It may be of interest to note that New Mexico has recently organized a large district, under the Conservancy Act of the State of New Mexico, which closely follows the Conservancy Acts of the States of Ohio and Colorado. Ohio and Colorado used the conservancy form of law to accomplish flood-control work. The Conservancy Act of New Mexico has been adapted to the purpose of accomplishing flood control, irrigation, and drainage. This experiment with a fifth agency has heretofore not been tried.

The project extends along the Rio Grande from the Elephant Butte Reservoir on the south, to White Rock Canyon on the north, a distance of about 150 miles. The district is known as the Middle Rio Grande Conservancy District. This District would fall into the general group of "A" Districts, considered by Mr. Henny, because the land values are greater than the face value of the proposed bond issues. The organizers of the District had in mind that the conservancy form of law was better adapted to the local problems than the irrigation district methods, for the following principal reasons:

(1) It provides for the "benefit method" of assessments, which insures that the municipalities and corporations, affected by the proposed work, will be required to pay a fair share of the cost thereof. The flexibility of this method of assessing costs is also well suited to take care of the great variation in local soil and irrigation conditions;

(2) The law grants wide powers to the district, including the police powers of the State;

(3) The preparation of an official plan is required prior to the authorization of the work by the District Court. This Court, sitting as a Conservancy Court, either approves or disapproves the plan, after holding hearings thereon. If the Court approves the plan, it then holds hearings on the appraisals, and when these are confirmed, the District may proceed to issue and sell bonds.

The work of the Middle Rio Grande Conservancy District has not proceeded far enough to offer proof of the superiority of this form of law for accomplishing reclamation problems, but the writer believes that many advantages are gained under such a law, especially where the contemplated work includes a variety of features.

Perhaps, one of the difficulties that has surrounded irrigation development, under all the agencies utilized to date, has been the limitation of the powers of that agency. Some degree of flood control and river protection is a common need in many of the irrigated valleys. Drainage is now known to be necessary for protecting practically all irrigation developments.

Drainage and flood-control works may directly benefit municipalities and corporations as well as lands, and the problems involved are properly a part of the general reclamation scheme, including irrigation. As a rule, these features are so connected and inter-related that they can only be solved in the most effective and economical way by considering them as a single problem. The Conservancy District agency seems to offer many outstanding

advantages for accomplishing reclamation work, involving irrigation, flood control, and drainage features. One of these is that all property that is benefited is taxed, not only the agricultural lands.

The success or failure of some irrigation developments to repay construction costs is judged entirely by an artificial financial status, which in no way represents the actual wealth created by the project. The time element of the repayment period has been an entirely arbitrary one, yet projects are classed as failures unless repayment is made within the period set. For these reasons, it may be pertinent to examine the question further before pronouncing sentence as to success or failure. Perhaps, if proper standards are set up, there will be more successful projects and fewer failures. Undoubtedly, some projects that are now pronounced as failures, will be adjudged successful after the early losses are absorbed.

Many of the private companies effecting irrigation work in the South Platte Valley, near Denver, Colo., became bankrupt in the early stages of the development. This District is quite successful at the present time, and the City of Denver, and many prosperous smaller towns depend quite largely on the wealth created by the irrigation of about 1 500 000 acres in this vicinity.

Furthermore, it seems evident to the writer that the recent depression in irrigated farm values is largely the result of the general slump in value of farm products. This slump has affected land values in all parts of the country; in humid areas as well as in irrigated sections.

The Federal Reclamation Acts provide that the total cost of the projects shall be repaid by the benefited agricultural lands. Many cities and towns have developed and prospered along with the irrigated farms as the direct result of Government irrigation. Among the largest of these are El Paso, Tex.; Phœnix, Ariz.; Yakima, Wash.; and Boise, Idaho. The taxable wealth in these cities, and the many smaller communities on reclamation projects, is in no way a resource of the project and, yet, irrigation is responsible for the major part of the wealth represented by them.

Another case of wealth, created by irrigation, is the profit taken by individual land holders who sell before repaying their construction charges. Large and even excessive profits have been taken in this way from practically all large irrigation developments. The purchasers may now be having difficulty in meeting repayment costs and interest on the high price paid for the land, but the seller is enjoying the full profit. It is the rule, on all Federal projects, that the lands are commonly bought and sold without any account being taken of the unpaid construction charges.

On the Rio Grande Project, New Mexico and Texas, certain flood-control work is now being done by the City and County of El Paso. It is doubtful whether the Government could have accomplished this work under existing laws, but it is a normal phase of the Government's work. The Elephant Butte Dam is affording protection against floods in the City of El Paso; yet, no charge could be made against the municipality.

The original Reclamation Act contemplates only the development of large desert land areas owned, in part at least, by the Government. If irrigation

development under Federal direction is confined to such areas, subsidies from the State or National Government are justifiable as offsets against the taxable wealth created by the development, which cannot be directly assessed for work accomplished. Federal development of desert land areas should be continued as fast as additional farms are in demand.

Federal reclamation has not been restricted to such developments, however, and, in some instances, the result of this has been an unequal distribution of the assessed cost of construction. If the Federal Government is to continue a program of reclamation that is suited to all the reclamation problems of the Western United States, the existing laws should be changed to permit greater flexibility and a wider scope of accomplishment. This should include drainage and flood-control work. The necessity of increasing the scope of Federal Reclamation may be open to question because many drainage and flood-control projects have land values greater than the cost of the proposed improvement. When this is the case, it is usually possible to finance work at favorable terms, and progress may thus be made through district organizations without help from the National Government.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

TIDES AND THEIR ENGINEERING ASPECTS

Discussion*

BY MESSRS. LEWIS M. HAUPT, H. DEB. PARSONS, R. L. FARIS, AND EARL I. BROWN.

LEWIS M. HAUPT,† M. AM. SOC. C. E. (by letter).‡—This comprehensive paper, by Commander Rude, describes one of the most important factors in the interchange of the world's products. His statement of the complex nature of this problem, created by agencies which are constantly modifying the operation of tidal elements, is prefaced by sixty-four definitions of its many phases. Celestial and terrestrial gravitation, temperature, light and darkness, hills and valleys, continents and islands, winds, waves, and currents, are all factors which affect tidal fluctuations.

"All the rivers run into the sea; yet the sea is not full; unto the place from whence the rivers come, thither they return again". This is due to celestial and terrestrial gravitation (formulated by Newton), yet it is the returning streams that deposit the bars at the mouths of the rivers, thus affecting transportation, limiting the interchange of products, and increasing its cost. It is, therefore, a vital factor in the general welfare of humanity.

It is also worthy of note that, although on some coasts the tides are very feeble and diurnal, in others they are of great amplitude and semi-diurnal while, as the author shows, they are ever varying and are affected by other terrestrial agencies such as winds, waves, and currents.

Another factor in navigation is the form of the channel and the velocity and sectional area of the vessel, causing displacement in canals or aqueducts and creating a crest in advance with suction in the rear, which seriously affect the banks of narrow channels. These, however, are not "tides", strictly speaking.

---

* This discussion (of the paper by G. T. Rude, M. Am. Soc. C. E., published in August, 1927, *Proceedings* and presented at the meeting of September 7, 1927), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., Physical Hydrography, Cynwyd, Pa.

‡ Received by the Secretary, August 16, 1927.

As the tides are important factors in the erosion, transportation, and deposition of the sandy barriers which enclose the coasts and imperil navigation, it is important that they be carefully analyzed and utilized as remedial measures, as has been done by the U. S. Coast and Geodetic Survey and many foreign authorities. It has been demonstrated by maritime engineers the world over that tides may be utilized so as to create ample channels across ocean bars for the largest vessels, but in many instances these channels are being maintained by dredging to remove the drift carried in by the tides from adjacent coasts.

As an instance of vital moment, attention is called to the re-entrant angle in the coast between Long Island and New Jersey, where the inlets on both flanks have been moving toward the bar covering the 7-mile gap between Sandy Hook and Coney Island. Its greatest navigable depth at Gedney's Channel was 21 ft. until the Ambrose Channel was dredged to meet the imperative demands of commerce, but no permanent works have been constructed to check this littoral drift from Long Island. The Jamaica Bay Bar has now crossed the 50-ft. basin and is encroaching on the existing channel. It is believed that the exhaustive report made by the Coast Survey under Professor Henry Mitchell in 1858-59 has not been published, but it is an important document looking to the creation of a permanent entrance to the City of New York. The problem has been under consideration for many years, and is quite feasible, but as yet has received no permanent solution, reliance being placed on dredging.

In this connection, and as a matter of record, it seems pertinent to cite a demonstration of the utility of tidal action in the creation and maintenance of an automatic channel which was inaugurated by Mr. Brewster Cameron* in 1899. He succeeded in persuading Congress to undertake the establishment of a navigable channel by constructing an offshore barrier to check the littoral drift and admit the 14-in. diurnal tides. All previous efforts had failed and the inlet has been abandoned, with the remains of the previous Government work lying across the bar at a depth of 3 to 8 ft. In 1906, after many delays, the detached offshore, curved or "reaction jetty", was reported to Congress to have been completed at a cost of only $389 203 and the Superintendent of the U. S. Coast and Geodetic Survey stated that the predicted automatic channel was in evidence, without dredging or bar advance. In January, 1910, it was desired to secure a depth of 25 ft., and it was proposed that a second jetty should be built and the offshore breakwater connected with the island. The cost was estimated at $2 028 125, with $60 000 additional to be used for annual dredging, which was not permitted under the former work, the automatic channel having been created by the 14-in. diurnal tides. A second jetty was constructed, however, and it reduced the depth immediately to 13.3 ft. at the outer end, where it was more than 25 ft., and diminished the tidal influx passing through this 2-mile entrance, thus reducing its efficiency.

* Mr. Cameron lost his life at Niagara Falls.

Again, in 1912, another project was submitted to secure greater depths by the extension of both jetties and by dredging. The estimate submitted* was "for $2 325 000 to be completed in three years", but the diurnal tides continued to deepen along the concave reaction jetty so that this sum was entirely saved.

This example indicates the relative efficiency of purely tidal scour, due to a curvalinear directrix so well presented by H. C. Ripley, M. Am. Soc. C. E., in his paper,† "Relation of Depth to Curvature of Channels", in which reference is made to this reaction breakwater, which he built under authority of Congress and which is believed to be the only one in existence.

The Mediterranean Sea is regarded as tideless, yet in certain bays there are tides equal to those of the Gulf of Mexico. The Bay of Rio de Janeiro, Brazil, is notorious for the "resacas" which rise to more than 100 ft. at times, depending on the winds.

These are only a few of the main factors imposed by the tides as they affect the problems of the engineer in the creation of channels or the reclamation of beaches so vital to the welfare of the Nation.

H. de B. Parsons,‡ M. Am. Soc. C. E. (by letter).§—The work being done by the U. S. Coast and Geodetic Survey, as described by the author, is most useful to the profession. Some years ago, the writer undertook a study of the tidal phenomena of the Harbor of New York,‖ and appreciated the help that he received from the Survey.

There are only a few engineers who comprehend tidal phenomena and the regimen of the rise and fall of tidal waters. Structures are frequently erected in the beds of tidal waters and streams, and, when they are, a comprehensive knowledge of tidal ranges and of current velocities is most essential. Many engineers are prone to consider the tidal rise and fall as being approximately harmonic, and do not make proper allowance for hydraulic conditions that affect the range and the irregularities of recurring high and low waters.

It is unfortunate that engineers and surveyors use so many different datums as reference planes for engineering works. The result has been confusing because it has made it difficult to co-ordinate surveys based on different datums. In many instances, the datum used is an arbitrary assumption, and then co-ordination is impossible. The establishment of precise mean sea-level datum bench-marks throughout the United States is a development of the work of the Coast and Geodetic Survey that engineers and surveyors in civil practice should utilize, as it will eliminate confusion and make it possible to co-ordinate surveys made at various times and under different conditions. The universal use of a standard datum would be especially beneficial for hydraulic developments, railroads, highways, water supplies, sewage disposals, river regulations, and flood protection works. These are often interstate problems, and the advantage of a standard reference datum, correlating all

* H. R. Doc. No. 1125, 62d Cong., 3d Session, December 11, 1912.

† *Transactions*, Am. Soc. C. E., Vol 90 (June, 1927), p. 207.

‡ Cons. Engr.; Prof. Emeritus, Rensselaer Polytechnic Inst., New York, N. Y.

§ Received by the Secretary, August 16, 1927.

‖ *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 1979.

adjacent surveys, must be evident. It frequently happens that questions relating to elevations arise in cases under litigation. Such cases would be clarified if a standard elevation had been used for the different surveys that are the bases of the questions under dispute.

The U. S. Geological Survey uses mean sea level as the datum for its topographical maps of the United States, as also does the U. S. Lake Survey for its studies of the waters of the Great Lakes. Perhaps, in the course of time, the U. S. Coast and Geodetic Survey will state the land heights on its charts as feet above mean sea level, instead of in feet above high water, or will print the correction on the charts. These charts and topographical maps are used extensively for preliminary studies of important engineering works.

In contributing this discussion, the writer has in mind the hope that engineers and surveyors will use mean sea level as determined by the U. S. Coast and Geodetic Survey when selecting their datums. When an arbitrary datum is in local use and cannot be readily changed, a note should be placed on drawings giving the correction required in order to refer the datum to mean sea level.

The writer was pleased to find that the engineers of the Board of Transportation of the City of New York had placed a note on a recent drawing, referring the arbitrary datum representing mean high water at New York, to mean sea level as established by the U. S. Coast and Geodetic Survey at Sandy Hook.

Engineers interested in erosion of beaches and in harbor improvement will find the records of coastal currents extremely enlightening. This work is being carried forward by the Coast and Geodetic Survey and data (supplemented by the records of the U. S. Weather Bureau) are being accumulated that will give good pictures of normal and abnormal conditions along the shores of the United States.

The public, as well as the profession, is indebted to the Engineers of the U. S. Coast and Geodetic Survey for the careful and painstaking work that is being done and published for general use.

R. L. Faris,* M. Am. Soc. C. E. (by letter).†—The paper brings out well the various aspects of tides and currents as they concern the engineer. As yet the engineer has not contributed to this subject as much as might be desired, notwithstanding the fact that as regards certain phases, for example the changes in tidal régime brought about by river and harbor improvement, he is in a position to make contributions of a fundamental character.

If any one branch of the subject may be regarded as of basic importance to the engineer, it is undoubtedly that having to do with datum planes, for they are a matter strictly within his domain. It is not so many years ago that it was customary for the engineer to start with some arbitrary bench mark as "datum" and refer all elevations within a given locality to this datum. If the bench mark originally defining this datum were lost, or its elevation accidentally changed, no means were at hand for re-establishing it

* Asst. Director, U. S. Coast and Geodetic Survey, Washington, D. C.

† Received by the Secretary, August 24, 1927.

accurately at its original elevation. Furthermore, with the growth of urban communities, characteristic of the past decades when lines of levels based on such arbitrary datums met, confusion was inevitable. Datum planes based on tides represent a step in the direction of standardization.

With the adoption of datums based on the tides the engineer is in a position to recover such reference planes even if all bench marks are lost. Furthermore, these tidal datums furnish natural, instead of arbitrary, bases, which are related to physical phenomena with the concept familiar not only to the engineer but to the intelligent public.

It is to be observed that while tidal datum planes possess the advantages of simplicity of definition, accuracy of determination, and certainty of recovery, their determination is not a simple matter. The laws governing the rise and fall of the tides are such that, together with the effect of wind and weather, any phase of the tide chosen as a datum plane, as, for example, mean sea level, mean high water, lower low water, etc., changes from day to day, from month to month, and from year to year.

Despite these variations, however, as is brought out by the author, methods have been developed to reduce to mean values the results of short series of observations. By the use of these methods, in general a datum plane may be derived from a month of tide observations correct to within 0.1 ft. and from a year of observations correct to within 0.05 ft.

Even limiting the matter to that which may be of concern to the engineer, the subject is one of wide extent. Necessarily, therefore, the author has been obliged to deal with the various phases of the subject in a brief manner. The full references, however, add to the value of the paper. With the improvements of tidal waterways which the growing water-borne commence will make necessary, the engineer will be called upon to solve problems that demand a knowledge of tides and currents. Such knowledge this paper gives.

EARL I. BROWN,* M. AM. SOC. C. E. (by letter).†—This paper is an excellent statement of present-day scientific knowledge of tides, and of the practical uses made of such knowledge. In reading it one cannot fail to be struck most forcibly by the thought that tidal phenomena are too much neglected in text-books on hydraulics. Papers like this go a long way toward supplying the deficiency. As the author points out, a practical working knowledge of the theory and application of tidal hydraulics is a necessity for any engineer engaged in the maintenance or improvement of channels in tidal waters. A lack of understanding of tidal mechanics has been responsible for many of the mistakes and failures made by engineers in planning and executing maritime works.

In practice, the engineer is not ordinarily concerned with the tides or currents in the open sea, far off the coast. He is interested in those adjacent to the coast, and in the inland waters, and it is these that will be understood in what follows.

* Col., Corps of Engrs., U. S. A.; Engr., Eighth Corps Area, Fort Sam Houston, Tex.
† Received by the Secretary, August 30, 1927.

There is not much in the paper to which the writer would take exception. It might be well to point out that the author's classification of waves into progressive and stationary waves is a purely arbitrary one, as there is no basic difference between them. All waves are fundamentally progressive, but when a progressive wave is reflected back upon itself at points one-half or one full wave length (or multiples thereof) from the origin, a stationary wave results from the interference of the components. The resultant wave derived from such a reflection will show characteristics more closely approaching those given as defining each class of wave, depending on how closely the point of reflection coincides with the half wave length or its exact multiple in distance from the origin.

As the author shows, the propagation of the tide from the sea into inland waters gives rise to so many apparently confused and conflicting manifestations of heights of water, and directions and velocity of currents that to bring order out of the chaos and to deduce general laws seems to be impossible. Nevertheless, it has now been found possible to account for almost all these phenomena by considering the tidal wave entering inland waters, as a progressive wave of translation, acted upon and modified by friction of the bed, depth and width of the channel, and combined with entire or partial reflections at those points where a change of energy of the wave is effected by the bed.

The author has classified the currents flowing through a narrow strait connecting two larger tidal bodies of water as being hydraulic currents; meaning by this term a current flowing under the influence of a surface slope.

The writer has had experience on many canals connecting such bodies of water along the coast, at points ranging from Delaware to Texas. He has also observed many inlets connecting the sea with large inland bays, in which the connecting channel was of varying lengths from a few hundred feet to several miles. He has always found the flow through such channels, when of considerable length, to be far from hydraulic. The surface slope is not uniform, nor are the current velocity and rate of discharge uniform for the whole length of the canal. All these phenomena vary from point to point, and the current may be strongest where there is no slope or even an opposed slope. As the connecting channel or canal becomes shorter, these manifestations of true tidal propagation are less evident, but still persist, and it is only when the canal is very short indeed that true hydraulic flow may be said to prevail. The author is correct, however, in stating that the simpler hydraulic solution of problems arising in some cases may suffice for all practical needs. The engineer must bear in mind, however, that such a solution is a mere approximation, and it should be used only for very short canals or straits.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

NOTES ON ARCHED GRAVITY DAMS

Discussion*

By Messrs. George R. Rich and A. Floris.

George R. Rich,† Assoc. M. Am. Soc. C. E. (by letter).‡—Curving a gravity dam up stream does not add to the stability of the structure unless the vertical contraction joints between sections are pressure-grouted to insure contact after the heat due to chemical action has been dissipated and the resulting shrinkage has occurred. The writer believes that the volume changes of the entire mass caused by seasonal variations in temperature are of small magnitude compared to that produced by the heat liberated during the chemical process of setting. Temperature measurements on sections of the Wilson Dam show that several months may be required for the dissipation of the chemical heat.§

Gravity dams of any appreciable length are ordinarily constructed in sections, with vertical contraction joints having shear keys and sealing strips, but without recourse to pressure grouting of the vertical joints. After the aforementioned initial shrinkage has occurred, there will be an opening between adjacent sections and before there could be sufficient deflection to give the contact prerequisite to action of the horizontal arch elements, the vertical cantilever elements would have to fail in tension near the heel of the dam. From this point action toward ultimate damage is progressive, owing to the effect of uplift pressure entering the tension cracks.

The author demonstrates conclusively that the conventional method of calculating curved gravity sections does not furnish an accurate indication of the existence of heel tension with reservoir full. This analysis, excluding the uncertainty of shear distribution at the foundation as a defense of the

* This discussion (of the paper by B. F. Jakobsen, M. Am. Soc. C. E., published in August, 1927, *Proceedings*, but not presented at any meeting), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion

† Hydr. Designer, Stone & Webster, Inc., Boston, Mass.

‡ Received by the Secretary, August 10, 1927.

§ "Control of Mixture and Testing of Wilson Dam Concrete," by John W. Hall, *Proceedings*, Am. Concrete Inst., Vol. XXII, 1926.

usual method of calculation, is a step toward more rational determination of stresses in hydraulic structures.

His derivation of the fundamental formulas has the advantages of clearness and simplicity. The writer's independent check of Equations (7)* and (8)† by direct successive integration, using semi-polar cylindrical co-ordinates instead of the usual three-dimensional system, may be of some academic interest.

Let p = the radius vector, θ = the variable angle, and z, the height axis. (See Fig. 3.) Taking the origin of co-ordinates at the apex of the conic surface that includes the down-stream face of dam, the equation of the conic surface is,

$$p = \frac{B\,z}{H}.$$

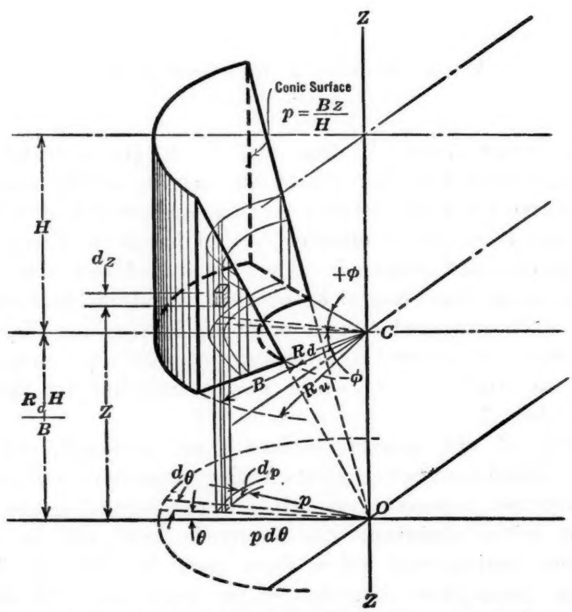


Fig. 3.—Deriving Formula for Conic Surface by Use of Semi-Polar Cylindrical Co-Ordinates.

From which the moment of the concrete about C (Equation (7)):

$$M_c = \int_{R_d}^{R_u} \int_{-\phi}^{\phi} \int_{\frac{R_d\,H}{B}}^{p\,\frac{H}{B}} w\, p^2 \cos\theta\, d\,p\, d\,\theta\, d\,z$$

$$= \frac{w \sin\phi\, H}{2\,B} (R_u^4 - R_d^4) - \frac{2\,w \sin\phi\, H\, R_d}{3\,B} (R_u^3 - R_d^3)$$

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1138.

† *Loc. cit.*, p. 1139.

Simplifying and substituting $R_u = B + R_d$, there results:

$$M_c = w \sin \phi \, H \, B \left(R_d^2 + \frac{4 \, B \, R_d}{3} + \frac{B^2}{2} \right)$$

The weight of the concrete,

$$W = \int_{R_d}^{R_u} \int_{-\phi}^{\phi} \int_{\frac{R_d H}{B}}^{p \frac{H}{B}} w \, p \, d \, p \, d \, \theta \, d \, z$$

$$= \frac{2 \, \phi \, H \, w}{3 \, B} (R_u^3 - R_d^3) - \frac{\phi \, H \, R \, d \, w}{B} (R_u^2 - R_d^2)$$

Simplifying and substituting $R_u = B + R_d$,

$$W = w \, \phi \, B \, H \left(R_d + \frac{2 \, B}{3} \right).$$

A. FLORIS,* ESQ. (by letter).†—The author has derived formulas for the calculation of the trapezoidal stresses in curved gravity dams of a triangular profile.

The influence of the curvature on the shape of the horizontal sections was commonly considered by careful designers. The Italian professors, Ganassini and Danusso,‡ in checking the design of the gravity base of the Gleno Dam, after its failure, have used similar expressions for the calculation of the trapezoidal stresses. The derivation of these formulas is given here in some detail.

Making (see Fig. 4):

$$d = a \, \lambda \quad \text{and } b = a + \mu \, d = a \, (1 + \mu \, \lambda)$$

the area of the section,

$$A = \frac{d}{2} (a + b) = \frac{a \, d}{2} (2 + \mu \, \lambda) \ldots\ldots\ldots\ldots\ldots\ldots (18)$$

The distance of the center of gravity of this area from the up-stream face is,

$$c_0 = \frac{d}{3} \, \frac{2 \, a + b}{a + b} = \frac{d}{3} \, \frac{3 + \mu \, \lambda}{2 + \mu \, \lambda}$$

and the moment of inertia of the section with respect to the axis, 0 - 0, has the value,

$$I = \frac{a \, d^3}{36} \left[3 + \mu \, \lambda + \frac{(1 + \mu \, \lambda)^2 - 1}{(2 + \mu \, \lambda)^2} \right] \ldots\ldots\ldots\ldots\ldots (19)$$

which is derived as follows: Using the well-known formula:

$$I_c = I_0 + A \, c^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

there is obtained for the two triangles and the rectangle, respectively, into which the total area, A, may be divided, the values,

$$I_t = \frac{a \, d^3}{36} \, \mu \, \lambda$$

* Los Angeles, Calif.

† Received by the Secretary, August 31, 1927.

‡ "Relazione peritale sopra la cause che hanno determinato la rovina della diga de plan di Gleno in Val di Scalve, crollata la mattina del 1° dicembre, 1923," *Annali* dei Lavori Publici, Anno LXII, Fasc 5, Maggio, 1924, pp. 427–428.

and,

$$I_r = \frac{a\, d^3}{12}$$

and for their sum the value,

$$I_0 = \frac{a\, d^3}{36}\,(3 + \mu\,\lambda) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

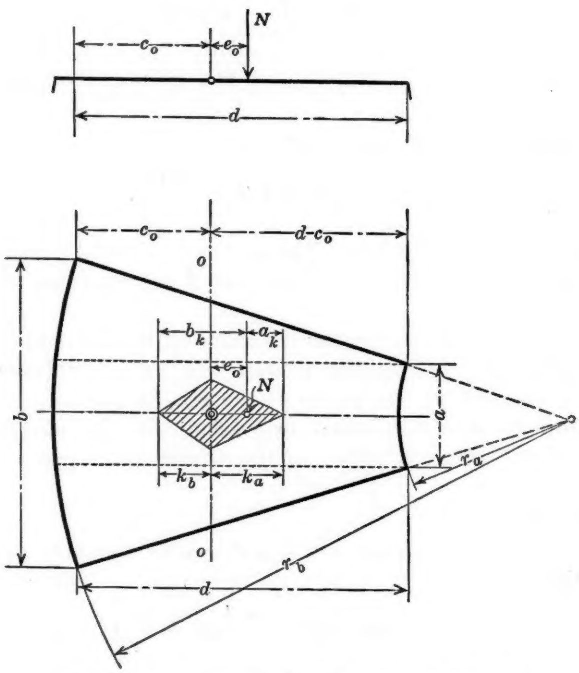


FIG. 4.—CALCULATION OF TRAPEZOIDAL STRESSES.

The distances of the centers of gravity of the partial areas from the center of gravity of the total area are given by,

$$c_t = \frac{d}{3} \cdot \frac{3 + \mu\,\lambda}{2 + \mu\,\lambda} - \frac{d}{3} = \frac{d}{3}\,\frac{1}{2 + \mu\,\lambda}$$

and,

$$c_r = \frac{d}{2} - \frac{d}{3} \cdot \frac{3 + \mu\,\lambda}{2 + \mu\,\lambda} = -\frac{d}{6}\,\frac{\mu\,\lambda}{2 + \mu\,\lambda}$$

while their corresponding areas are,

$$A_t = \frac{a\,d}{2}\,\mu\,\lambda \text{ and } A_r = a\,d \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

so that there is,

$$A_c{}^2 = \frac{a\,d^3}{36}\left[\frac{2\,\mu\,\lambda}{(2 + \mu\,\lambda)^2} + \frac{\mu^2\,\lambda^2}{(2 + \mu\,\lambda)^2}\right] = \frac{a\,d^3}{36}\,\frac{(1 + \mu\,\lambda)^2 - 1}{(2 + \mu\,\lambda)^2} \ldots (23)$$

The sum of the values of Equations (22) gives Equation (18), while the addition of Equations (21) and (23), according to Equation (20), results in Equation (19).

The stresses at the down-stream and up-stream faces are given, respectively, by,

$$\left.\begin{aligned} \delta_a &= \frac{N}{A} + \frac{N\,e_0}{I}\,(d - c_0) \\ \delta_b &= \frac{N}{A} - \frac{N\,e_0}{I}\,c_0 \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots (24)$$

Because, as the author correctly states, the middle-third theorem does not hold for curved dams, it may be appropriate to introduce the core theory (German kern) in the stress analysis of such structures. This gives a simple means of knowing whether or not tension does occur, because by excluding tension the resultant of all the forces must fall within the core.

The two points of the core on the line of symmetry of the section are given by the core radii:*

$$k_b = \frac{I}{A\,(d - c_0)} \text{ and } k_a = \frac{I}{A\,c_0} \quad \ldots\ldots\ldots\ldots (25)$$

Solving for I and putting these values into Equation (24),

$$\left.\begin{aligned} \delta_a &= \frac{N}{A}\,\frac{k_b + e_0}{k_b} = + \frac{N}{A}\,\frac{b_k}{k_b} = + \delta_0\,\frac{b_k}{k_b} \\ \delta_b &= \frac{N}{A}\,\frac{k_a - e_0}{k_a} = \pm \frac{N}{A}\,\frac{a_k}{k_a} = \pm \delta_0\,\frac{a_k}{k_a} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots (26)$$

If $e_0 = k_a$, then $\delta_b = 0$, while when $e_0 > k_a$, there is tension at the up-stream face and δ_b is negative.

Equations (25) and (26) are general expressions and valid for any section, provided the proper values for A and I are introduced.

In case the dam is straight in alignment, the horizontal section is a rectangle, and $b = a$. Consequently,

$$A = b\,d$$

$$I = \frac{b\,d^3}{12}$$

and,

$$c_0 = d - c_0 = \frac{d}{2}$$

and Equations (25) become,

$$k_b = k_a = \frac{d}{6} \quad \ldots\ldots\ldots\ldots (27)$$

It will be seen, therefore, that the middle-third theorem is merely a special case of the more general core theorem. With the values of Equation (27), Equations (26) takes the form,

$$\delta_a = +\,6\,\delta_0\,\frac{b_k}{d} \text{ and } \delta_b = \pm\,6\,\delta_0\,\frac{a_k}{d}$$

in which, if $e_0 > \frac{d}{6}$, δ_b becomes negative.

---

* "Die graphische Statik der Baukonstruktionen," H. Mueller-Breslau, Erster Band, 4 Auflage, 1905, p. 78.

The author's Equation (2)* for rectangular sections was given by Link†, many years ago, in his book on gravity dams of the triangular type.

The question whether inclined or horizontal sections should be considered in the calculation of the trapezoidal stresses has been the subject of much discussion in recent years. Professor Résal,‡ using elementary methods, devotes nine pages in order to show the necessity of investigating inclined sections. His main reason was that the horizontal sections are not perpendicular to the bisectrix of the angle, formed by the up-stream and down-stream faces at the crest of the dam. Professor Résal concludes therefrom that the usual bending formulas for prismatic bars are no more valid for beams of triangular shape. In order to correct this discrepancy, he suggested the assumption of the bisectrix as the neutral axis, taking the sections under consideration normal to it.

Dr. Gebauer§ also recommends the investigation of these normal sections because, as he claims, tension may occur at the up-stream face of the dam, while stress investigations for horizontal sections fail to detect this. He states, further, that the stresses, thus computed, for gravity dams might be 35% higher than those found by considering horizontal sections.

The use of inclined sections can only be justified, if the dam is analyzed according to the old theory, in which the determination of the trapezoidal stresses alone was considered to be sufficient. However, if the principal stresses are calculated, the use of inclined sections is not necessary. Assuming "plane strain" and the validity of the trapezoidal law, Dr. Kelen‖ has shown that for triangular profiles the same principal stresses are obtained regardless of whether horizontal or inclined sections are used. Inasmuch as the stress computations are much simpler for horizontal sections, most designers will prefer this method of procedure.

---

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1136.

† "Die Bestimmung der Querschnitte von Staumauern und Wehren aus dreieckigen Grandformen," E. Link, Berlin, 1910; see, also, M. Foerster, "Taschenbuch für Bauingenieure," Berlin, 1923, p. 1231.

‡ "Formes et dimensions des grands barrages en maçonnerie," *Annales des Ponts et Chaussées*, II, 1919.

§ *Beton und Eisen*, 1924, Heft 19.

‖ "Die Spannungsverhaeltnisse in Staumauern," Dr. Ing. N. Kelen, *Beton und Eisen*, 1925, Heft 18, pp. 287–290; also, "Die Staumauern," by the same author, Berlin, 1926, pp. 97–101.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

PREPARING THE GROUNDWORK FOR A CITY: THE REGRADING OF SEATTLE, WASHINGTON

Discussion*

By Charles E. Fowler, M. Am. Soc. C. E.

Charles E. Fowler,† M. Am. Soc. C. E. (by letter).‡—The re-grading of Seattle, as carried out largely under R. H. Thomson, M. Am. Soc. C. E., and the author, approached in magnitude of the material to be handled, the digging of the Panama Canal, and in some respects the difficulties were much greater. The first extensive re-grading was done by steam shovels on Denny Hill near the old Washington Hotel, the material being hauled in dump cars, down the principal business streets, to the tide flats for making fill. The writer operated two steam shovels in this general district, one at the Washington Hotel, and one on the old University grounds on Fourth Avenue. The material was sold to the Great Northern Railway for tide-flat filling.

The steam shovels were served by horse-drawn, $2\frac{1}{2}$-yd. dump wagons, from 24 to 30 for each shovel, and the dirt was hauled about $\frac{1}{2}$ mile to the north end of the Great Northern Railway Tunnel, where it was chuted into railway dump cars about 30 ft. below, and hauled in trains to the final place of deposit, south of the Great Northern Station. The selling price was 7 cents per cu. yd., which, added to the excavation price, made the total receipts about 20 cents per cu. yd., or a very profitable figure for the pre-war period.

The dumping grounds were just east of the territory and comprised several hundred of the 1 200 acres of the Seattle tide flats, which was brought to grade at 2 ft. above extreme high tide. This filling was done by the writer with two 20-in. hydraulic dredges, and the 12 000 000 cu. yd. so placed, formed an essential part of the re-grade work of Seattle, making usable the land from the foot of Queen Anne Hill to the southern end of Elliott Bay.

* This discussion (of the paper by Arthur H. Dimock, M. Am. Soc. C. E., presented at the meeting of the City Planning Division, Seattle, Wash., July 15, 1926, and published in August, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., New York, N. Y.

‡ Received by the Secretary, August 23, 1927.

The portion of the tide flats near the Jackson Street re-grade was filled with the dirt sluiced down in doing this work. This, together with the material removed from the Denny Hill re-grade, constitutes the greatest sluicing operation ever carried out, and it redounds to the credit of those who did the work, as well as to the City of Seattle, the possibilities of which were dependent on the great works described.

Not less notable in the making of the City that had the courage to re-grade its fills, fill in its tide flats, and build its port, was the creation of the Park System. This work was planned and begun while the writer was a member and, part of the time, President of the Park Board. Parkways, now about 60 miles in length, connect the city parks proper and represent a form of regrading of the hills of the city so as to make the beautiful drives, which wind about the hills and through much of the primeval forest preserved for the pleasure of future generations. This system is certainly a wonderful creation of man and is the result of allowing full play to the imagination and creative energy of the engineer. Hundreds of thousands will thank the imagination and tireless energy of Mr. Thomson who for more than a score of years was City Engineer and the creator of the future of the city.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

RE-ARRANGEMENT OF A BUSINESS DISTRICT: CHANGES IN RECENT YEARS IN PITTSBURGH, PENNSYLVANIA

Discussion*

By Edwin K. Morse, M. Am. Soc. C. E.

Edwin K. Morse,† M. Am. Soc. C. E. (by letter).‡—When it is realized that the area of the central business district of Pittsburgh is only 218 acres; that Chicago contains 600 acres within "the Loop"; Philadelphia, 1 400 acres; Boston, 1 900 acres; and New York City, to 42d Street, 4 000 acres, the comparison impresses one with the intensity of the Pittsburgh business man's activity, because, in no equal area in the civilized world is the same volume of business so successfully transacted. The Flood Commission of Pittsburgh has recently presented comprehensive plans for wharf and river improvements as prepared by its Board of Engineers in April, 1925, which, if approved by the U. S. Army Engineers and carried out by the City, will add more toward maintaining Pittsburgh's supremacy than any other local construction.

By deepening the channels of the Allegheny and Monongahela Rivers to a uniform depth of 16 ft., and straightening their shore lines, it will reclaim about 57 acres of what is now a sloping wharf of little value to a small shipping interest, costing practically nothing to the City.

The plans contemplate modern slips for river and rail transportation. They specify walls around the city and along the North Side to a level above the highest known flood. The surface and basements of more than 40% of the 218 acres in the central business district were damaged during the disastrous flood of March 15, 1907.

The contemplated construction would bring about the rebuilding of all the flooded districts in the City, and in ten years would completely repay the City

* This discussion (of the paper by Nathan Schein, M. Am. Soc. C. E., presented at the meeting of the City Planning Division, Seattle, Wash., July 15, 1926, and published in August, 1927, *Proceedings*), is printed in *Proceedings,* in order that the views expressed may be brought before all members for further discussion.

† Pittsburgh, Pa.

‡ Received by the Secretary, August 20, 1927.

by increased taxation of the section benefited. The 57 acres would be turned over to the Planning Commission of the City of Pittsburgh for comprehensive study. A boulevard, 80 to 100 ft. wide and more than 2 miles long, with no streets crossing it, could then be built along the banks of the Allegheny and Monongahela Rivers. Parking space for thousands of automobiles would be provided in a location convenient to the business sections, and when the time comes, a free right of way for a double-deck street, involving no consequential damages, would be awaiting posterity.

The average yearly loss to Pittsburgh, caused by floods, is estimated at $2 000 000. This money loss could be greatly reduced if the Flood Commission's plans were adopted. If that Commission's plans for the improvement of the Allegheny, Monongahela, and Ohio Rivers, within the city limits, had been fulfilled as recommended, the level of the disastrous flood of March 15, 1907, would have been reduced at least 10 ft. and, instead of nearly $6 000 000, the City would have lost less than the present annual cost of $2 000 000. With provision for intercepting sewers, back-fill, re-grading of streets, and paving complete, the engineers of the Flood Commission estimate the cost of building the walls around the city and along the North Side at $6 000 000.

The Mayor and Council of Pittsburgh have authorized the Department of Public Works to report on the plans submitted by the Commission and submit plans for river terminals. In connection with this study, the City has just completed soundings in the Allegheny and Monongahela Rivers within municipal limits.

The U. S. Government expects to complete the locks and dams on the Ohio River from Pittsburgh to Cairo, Ill., a distance of 967 miles. That will provide an all-year river transportation. Pittsburgh, strategically located for river and rail transportation, has no river terminals. The City's study contemplates making Pittsburgh the greatest inland river terminal in the United States except New Orleans, La., which is ocean and river combined. The City will have to condemn whole sections of improved property along the river-front so as to provide adequate terminals for slips handling river tonnage and rail and river shipments. In addition to the moderately equipped docks and slips, locations are being considered for large basins in which to store floating crafts and arrange transshipment of barges between down-river and up-river ports.

The mills, factories, mines, railroads, etc., that direct traffic to the rivers, have been modernized to the limit. Nothing has been done by the City to modernize the rivers and provide river terminals. Within recent years, the big steel industries have built river steamboats and steel barges that are now sending thousands of tons of structural steel and steel products down the Ohio and the Mississippi Rivers as far as New Orleans. The Inland Waterways Corporation is operating successfully on the Mississippi and Warriors Rivers. It is hoped and believed that Pittsburgh will build modern river terminals, thereby profiting by the great tonnage received and dispatched.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE PLANNING OF THE INDUSTRIAL CITY OF LONGVIEW, WASHINGTON

Discussion*

By B. L. Lambuth, Esq.

B. L. Lambuth,† Esq.—Virtually all outstanding real estate development is a product of the joint effort of skilled men drawn from many professions. The modern tendency is for the realtor, the town planner, and the civil engineer to work together toward a desired end. One of the greatest sources of interest and pleasure in connection with the Longview development was the harmonious relations between a considerable group of specialized experts, including, in addition to those mentioned, architects, lawyers, transportation men, title experts, advertising men, and others.

An interesting feature of the operations at Longview to those interested in this type of development is the fact that in theory at least it has been possible to avoid a tremendous aggregate annual loss and wastage arising as a result of the necessary reconstruction at the center of American cities which necessarily accompanies growth by a process of accretion at the circumference. The founders of the city had adopted what has been designated by Mr. J. C. Nichols, of Kansas City, Mo., as a "skeletal" plan of development, whereby the entire city site is completely planned and zoned and the actual development of each district is started in a small way in its proper relationship to a finished whole. By this means it is expected that large economies will be affected. Although on the one hand some extra cost is necessarily involved in tying the various districts of the city together by suitable street improvements and public utilities, yet, on the other hand, the great bulk of the property is carried until required for development as vacant agricultural acreage with correspondingly low overhead carrying charges, such as interest and taxes. Further-

* This discussion (of the paper by S. Herbert Hare, Esq., presented at the meeting of the City Planning Division, Seattle, Wash., July 15, 1926, and published in August, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Mgr., Real Estate Dept., The Longview Co., Longview, Wash.

more, the plan adopted offers the advantage of permitting one of the several districts of a city to be prepared for development in relatively small parcels as actually needed for immediate use from time to time; but the principal advantage of the "skeletal" plan is that business, housing, institutional, and, in fact, all building development is in its proper relationship to a finished whole. Therefore, the usual necessity of tearing down and reconstructing building—improvements which ordinarily accompany growth of a city from nothing to 40 000 or 50 000 population—is quite largely obviated.

Another interesting feature of the Longview plan is the association of suburban agricultural and garden tracts with the other districts of the city, and as a part of the general scheme. This is on the principle that one of the very important and very substantial elements of urban population is the family seeking a small tract of land as a means of providing over-time employment and a home that is partly self-sustaining.

A large number of people visit Longview each year. Apparently, the first three outstanding impressions of the more thoughtful and better trained of these visitors are, first, that they are viewing a city in the making; second, that the city was planned before construction was begun; and, third, that beauty and spaciousness of design and execution have been attained.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

MODERN CRUSHED STONE AND GRAVEL SURFACING

Discussion*

By Messrs. Roy A. Klein, C. H. Purcell, and O. E. Stanley.

Roy A. Klein,† Assoc. M. Am. Soc. C. E. (by letter).‡—The writer agrees that crushed stone and gravel surfacing will continue for some time to be the dominant type of surfacing for the vast mileage of rural highways of the nation.

As to the selection of the most serviceable types of road surface, the saving in operation effected by the construction of higher types does not accrue to the agency that performs the work, but rather to the people who operate motor vehicles. Therefore, since the agencies that build the highways do not operate them, it is obviously impossible to plan highway improvements based on any theory of pure economics. However, since the highway engineer represents the public in the last analysis, he should plan to build and maintain carefully the best type of surface that the funds secured by taxation will provide.

It is difficult to determine the exact amount of motor-vehicle traffic or tonnage at which it is feasible or desirable to change to a higher type of surfacing, because there are a number of different factors that affect local conditions. Stage construction, that is, building standard graded earth road first, crushed rock or gravel surface next, and then pavement, as the demands of traffic increase, is a logical and economical program. When the crushed rock or gravel surface is worn out, the road-bed will have become well stabilized and the remainder of the surfacing material will form a good base for the pavement. On the other hand, if the demands of travel do not justify pavement, a re-surface can be added at a fraction of the original cost; thus,

* This discussion (of the paper by J. W. Hoover, Esq., presented at the meeting of the Highway Division, Seattle, Wash., July 15, 1926, and published in August, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† State Highway Engr., Salem, Ore.

‡ Received by the Secretary, September 21, 1926.

in either case, the annual cost for the use of the road surface is a relatively small amount.

The importance of foundations has been stressed by the author, and that is a point well taken. When there is any question as to the strength of the sub-grade, the best insurance is to provide a sub-surface course of large stone, crusher run, of 4 in. maximum size, in a layer ranging from 4 to 12 in., depending on conditions. The use of large base stone on sections where the soil shows excessive capillary action has been quite successful in Oregon, the larger voids between the rocks breaking up the capillarity. On sections where these foundation courses have been used, the maintenance cost of the surfacing has been considerably less than on other sections constructed under similar conditions, because the surface does not break through in the spring.

The construction of the thickened edge and the thinner center for macadams as well as pavements appears logical and is quite simple. The edge has always been the point of weakness of the "feather-edge" macadams; particularly under softened sub-grade conditions. The thickened edge will make it possible to maintain the full designed roadway width without loss by wearing away on the sides. The Oregon State Highway Department has several thickened-edge macadams completed and the results are very satisfactory.

On main traveled roads, the surfacing should not be less than 18 ft. wide and preferably 20 ft. The greater width, of course, permits and encourages the traffic to spread over the entire road rather than confine itself to a single rut, thus reducing maintenance cost.

In reference to the selection of materials and the desirability of hardness of materials or resistance to wear, it may be said that the general practice is to classify materials by the standard abrasion tests. It should be remembered, however, that all abrasion tests are made on dry materials, while material in the road is subject to the greatest wear in wet seasons. In a recent test, made in the Oregon State Highway Department Laboratory wet material showed an abrasion four times as great as dry material.

A few years ago, it was standard practice to provide a thick layer of fine materials in the top course under the principle of providing a loose mulch which could be constantly kept in shape by blade graders. Intensive maintenance was necessary under these conditions to prevent corrugations from forming. This intensive maintenance, while it rendered the road smooth, resulted in a dusty condition that was very disagreeable and actually hazardous when the road carried much traffic. Many experiments were made to set up the top surfacing with clay and produce a hard, compact surface. These tests were quite successful and, in turn, led to the use of an oil covering on the clay-bound surface.

At first glance the value of fine material would appear to be minimized by the use of oil since it would no longer be moved about by the blade machine. It would then seem that the old specification of water-bound macadam, using the larger stones, might well be adopted. However, there are several important reasons for not using larger stone: First, there would be little economy, because the improvement in crushing equipment produces the smaller sizes at practically the same cost; second, experience indicates that it is neces-

sary to scarify an oil-treated road at intervals and if the larger stones are used, the scarifying is more expensive and the cost of resetting the materials greater; and third, it is difficult to obtain a smooth, even surface, true to grade, crown, and cross-section, with coarse materials. For these reasons there should not be less than 2 in. of compacted materials, ranging from ¾ in. to dust, in the top course. The base course material should range in size from 1½ in. to ¾ in., both courses being well graded between limiting sizes.

The practice in Oregon is to mix all courses with clay thoroughly with the blade machine, using the ridge method, the quantity of clay varying between 20 and 40%, depending on the quality. Care should be taken to secure a slow-slaking clay with a high tensile strength. In some instances, in order to secure the best clay, it has been found necessary to haul it a greater distance than the crushed rock. However, many pits and quarries contain a good natural binder by which it is possible to procure a compact surface without the use of clay. Surfacing, bound up with clay, will not be ready to receive an oil treatment until after it has gone through a winter season.

Mr. Hoover has stressed the necessity of having a smooth sub-grade before beginning construction. Generally, a rough surface is caused by a rough sub-grade, and it is very expensive to take out the irregularities on the last course of surfacing by using fine broken stone. The necessity for proper maintenance during construction is also very apparent. The Oregon specifications state the kind and minimum amount of equipment that the contractor must furnish for the construction of a crushed rock or gravel surfacing project.

No mention has been made of the desirability of crushed gravel as compared with screened materials, but the advantage is so obvious as to preclude discussion. It should be the constant aim of all highway engineers to try to discover, by trial and experimentation, better methods of building these crushed rock and gravel road surfaces, commonly called macadams. The field is large and the opportunity great.

C. H. Purcell,* Assoc. M. Am. Soc. C. E. (by letter).†—Attention has been called by Mr. Hoover‡ to the large amount of Federal Aid money that has been expended on crushed stone or gravel construction. The fine-rock road is the contribution that engineers have made to the solution of automobile transportation problems. It might be well to emphasize further the important place that this type of road occupies in the development of a connected system of highways.

The financing of these highway projects has compelled engineers to make thorough studies of the crushed-rock road as a means of providing a large mileage as quickly as possible. Mr. Hoover is one who is faced with this problem and his paper brings the facts out clearly.

The great distances between towns, the necessity for roads that will carry traffic at high speed every day in the year, and the limited finances available

---

* Dist. Engr., U. S. Bureau of Public Roads, Portland, Ore.

† Received by the Secretary, September 21, 1926.

‡ *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1185.

to meet the increasing demand for mileage, have resulted in this pioneer type of road. It occupies a more important position in the West than in the wealthier and more densely populated sections of the country. There it has revolutionized travel and taken care of the great increase in motor transportation.

The fine-crushed rock road is peculiarly adapted to the region between the Rocky Mountains and the Pacific Ocean. It can be laid at low costs because of the quantity production of crushed rock and gravel. Suitable road material of various kinds is available practically throughout the entire section.

The engineer's problem has been to develop a type of surfacing that could be cheaply and rapidly built, and that could be easily maintained during the dry summer period when traffic reaches its peak. The success of this type in developing traffic has brought its own problems. Dust prevention is one of the most important construction questions confronting highway engineers in the West, for dust means a loss of road metal, discomfort, and perhaps danger to the motorist.

The suggestion made by Mr. Hoover that a section following pavement design more closely, and providing an increased thickness at the edge is one step farther toward designing this fine crushed rock road so that it can serve in the future as one stage of a higher type of road. If properly constructed and maintained, solving the loss of metal, it will serve as a base. For a future 18-ft. surface it would be better if the base were extended out at least 1 ft. on each side beyond the proposed top surface. The tendency (under maintenance) to increase the crown of fine-crushed roads has produced an unsatisfactory base for future surface. When preparations are made to put on a top surface the necessary scarifying to reduce the crown often destroys the bond between the rock at the center, and sometimes a thin base is the result.

The early fine-crushed rock roads in some of the Western States were designed with a feather-edge section. All recent construction is built in a trench section, thus securing a thicker edge. This change was brought about by the desire to have sufficient thickness at the edge to support the concentrated traffic. Experience with surface treatment has shown the importance of the thickened edge. For example, many of the roads in Oregon that have been surface-treated were originally constructed with feather edges. The relatively high cost of maintenance at the edges as compared to other portions of the surface has led to the change. Mr. Hoover's suggestion for a thicker edge is worth considering. In the West where long distances were surfaced with fine rock, many of the roads were built only 12 ft. wide and the road widened out, under maintenance, into a two-way road. It became necessary to re-shape these roads and add more rock before the surface could be treated.

In New England, where surfacing of the penetration type has been developed over a long period of years, old bases of water-bound macadam have been available. The fine-crushed rock roads in the West, built of sufficient

width and maintained at a proper thickness for a base (9 in. loose) will carry a penetration surface under the moderate traffic encountered. In fact, such roads, correctly built, can be designed to serve as the first step in the evolution of several better types. If they have a carefully processed top with a moderate amount of sand-clay binder, they can be given the oil treatment. Later, when the need requires it, they can be finished with an asphaltic penetration surface; and, finally, as the traffic becomes heavier, this may be followed by even higher types of surfacing.

Many of the fine-crushed rock roads have been constructed with either a 4-in. or a 5-in. bottom course and a 4-in. top course. The State of Oregon has been a pioneer in oil treatment of fine-crushed rock roads. Since the development of asphalt surface treatment, it has been building a thicker base course and a thinner top course. This change has only been made, of course, in cases where it was reasonably certain that some plan of surface treatment was to be applied as soon as the surface had become sufficiently compacted under maintenance.

The author has pointed out one of the very essential items that enter into a successful fine-crushed rock road, namely, the matter of uniform distribution of material. No matter how well the present modern field crushing plants are designed, there seems to be a lack of uniformity of grading between the different truck loads as they are deposited on the road. It is also difficult to distribute the filler or binder evenly over the surfacing material without adopting some means of mixing them after they are spread. The excellent results of surface treatment noted by the author in the section east of the Cascade Range, were due in no small measure to the attention given to this detail of construction. Great care was taken to insure a uniform mixture of material. Wherever water was available, the material was thoroughly soaked while being spread and processed on the road. If the surface is not wet down a large percentage of the binder is fanned out by the traffic and lost before the top has a chance to set up. As in the higher types of road, the most successful results can only be obtained by strict attention to the details of design and construction.

Too much stress cannot be placed upon the fact that actual service to the public is the determining factor. It is the duty of the highway engineer to emphasize continually the need of providing funds sufficient to furnish the public, the highest possible type of service. The traffic volume is so large on many roads that the maximum expenditure will be warranted on the basis of service value to the traffic, and the saving in operation costs.

Many State and Government agencies have investigated the saving of cost of tire and gasoline consumption on the various types of pavement. The writer does not know of any investigation of the depreciation on automobile engine, body, and other mechanism, due to the jolting on a rough, unimproved, earth road. It may be learned that the saving in rubber and gasoline is a small sum compared with this other greater loss to the automobile owner. If this is the case, the fine crushed-rock road of the West, which has been extended so rapidly at the moderate cost of from \$3 500 to \$6 000 per mile,

shows a remarkable saving. This accomplishment of the engineer, although not receiving as much attention as the paving, is far greater.

O. E. STANLEY,* M. AM. SOC. C. E.—In Portland, Ore., there are about 200 miles of old macadam streets, built 10 to 20 years ago.

These streets are maintained by reshaping, when necessary, with a steam roller, and giving them a coat of oil. After the road is shaped to the proper cross-section (which the foreman gets with his eye, using the curbs as guides), it is sprinkled and rolled until the fine material, or "soup", is brought to the surface. The road is then left open to the traffic for probably two weeks until the fine material dries and blows away.

There are a great many complaints about dust, but as soon as the dust has worn off and the surface of the rock is exposed, oil is applied with very good results; these roads remain in good condition from 1 to 3 years. When little holes wear in the surface, after about this length of time, they are patched with a mixture of asphalt and crushed rock. Sometimes only two or three holes occur in a block.

With this oil treatment, results are obtained that make some of the contractors wish the process had not been discovered.

* Chf., Bureau of Maintenance, City of Portland, Portland, Ore.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

STREAM POLLUTION IN THE PACIFIC NORTHWEST

Discussion*

By William J. Roberts, M. Am. Soc. C. E.

William J. Roberts,† M. Am. Soc. C. E.—The author upholds high ideals, and many of them are worthy of heartiest support. It has, however, been demonstrated beyond question that there are places on Puget Sound where it is neither necessary nor expedient to follow the author's advice‡ "to treat all sewage flowing into the Sound and to sterilize it to the satisfaction of the State Board of Health."

An outstanding example is furnished by Camp Lewis, American Lake Cantonment. As Consulting Engineer, collaborating with the Constructing Quartermaster, the speaker found that the water supply and the proper disposal of sewage for 50 000 troops presented many intricate problems. The demand for healthful conditions was no more imperative than the demand for speed in construction; and always the question: "Is it sanitary?" was followed by the remorseless demand: "Can the work be completed on time?"

Fortunately, "Sequalichew Springs" about 1.5 miles from Camp Headquarters, proved adequate in quantity, yielding, when developed, more than 6 000 gal. per min. The quality of this water was pronounced, after frequent tests, to be "as pure as distilled water." Thus, the problem of obtaining an adequate and pure water supply was solved.

The logical disposal of the sewage was into Puget Sound 3.5 miles from Headquarters, without treatment. After a thorough investigation, and careful consideration of all possible objections, the point of discharge was selected at mean sea level, 3.5 miles from the nearest barracks; more than a mile from the nearest dwelling, but only a short distance from a double-track railroad carrying a heavy passenger traffic. The tidal range of the

* This discussion (of the paper by the late William F. Allison, M. Am. Soc. C. E., presented at the meeting of the Sanitary Engineering Division, Seattle, Wash., July 15, 1926, and published in August, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., Tacoma, Wash.

‡ *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussion, p. 1210.

Sound at this point is about 12 ft. At the outlet the channel is about 1.5 miles wide and several hundred feet deep. Thus, the shores of the Sound are thoroughly cleansed twice a day by the large volumes of flowing tides. So thoroughly is this cleansing process carried on that a passenger on the train can not discover by sight or odor, the outlet of the sewer. No inconvenience or harmful effects of this plan of disposal touch the lives or even the comfort of the soldiers at Camp Lewis nor its environment. If ever there was a place where disposal by dilution and dispersion is permissible, because rational, then this is that place. The demand for perfect sanitation and great speed in construction were satisfactorily met; 40 miles of water mains and 30 miles of sewers were actually laid in 78 days.

It is estimated that the tortuous channels of Puget Sound with all its inlets and islands within the State of Washington amount to 3 600 miles of shore lines. The volume of ocean water pouring into Puget Sound through the Strait of Juan de Fuca, 16 miles wide, with a maximum channel depth in excess of 1 000 ft., assures a satisfactory dilution and dispersion of any sewage discharged into the Sound.

The Sanitary Field Officers at Camp Lewis, remembering the troubles occasioned in some of the camps during the Spanish-American War, were alert to avoid anything unsanitary. Hence it was a pleasure to learn from the best information obtainable that not a single case of typhoid fever could be traced to impure water at the Camp or to bad sewerage during the entire training period from August, 1917, to November, 1918.

As has been well stated:*

"Skimping funds may often lead to danger to the public health, and the enforcement of requirements for an unnecessarily high degree of purification of sewage to useless waste of money."

* Preface to "American Sewerage Practice," Vol. III, "Disposal of Sewage," by the late Leonard Metcalf, M. Am. Soc. C. E., and Harrison P. Eddy, M. Am. Soc. C. E.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE LAKE WASHINGTON SHIP CANAL, WASHINGTON

Discussion*

BY MESSRS. ERNEST B. HUSSEY AND CHARLES E. FOWLER.

ERNEST B. HUSSEY,† M. AM. SOC. C. E. (by letter).‡—The treatment of the subject of selecting one of five possible routes for the location of a ship canal to connect Lake Washington with tide-water is a valuable digest of the varied and interesting events that finally culminated in the adoption of the route chosen. With this phase the writer was identified in a consulting capacity and had occasion to go into the subject somewhat exhaustively. The final selection by the Government and the subsequent operation of the canal leaves no doubt but that the right location was adopted and that future generations will sustain that judgment.

The lowering of the surface of Lake Washington to that of Lake Union, thereby eliminating a lock between the two lakes, was unquestionably a sound plan; not especially from the point of economy in construction, but more particularly from that of efficiency in transportation. Further, Lakes Union and Washington become a prospective great fresh-water harbor for the commerce of Seattle and its hinterland.

While cut-off walls were placed 4 ft. deep under the heels of the walls of the locks and under the miter-sills, no valves or other devices were provided to relieve possible hydrostatic pressure. The fact that holes drilled through the 3-ft. floor of the large lock showed a very small quantity of accumulated water indicates how well the work of installation of the cut-off walls was accomplished. However, it is the writer's opinion that measures should be adopted in such construction to relieve automatically any hydrostatic pressure that may develop. The use of drains from the areas protected by the cut-off walls, freely discharging into sumps readily accessible for observation, is a measure of precaution well worth its cost in works of such importance,

* This discussion (of the paper by W. J. Barden and A. W. Sargent, Members, Am. Soc. C. E., presented at the meeting of the Waterways Division, Seattle, Wash., July 15, 1926, and published in August, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Civ. Engr., Seattle, Wash.

‡ Received by the Secretary, December 3, 1926.

particularly as the heavy floor might be subjected to 53 ft. of hydrostatic head when the lock is unwatered. Much reliance may be placed on the hard impervious clay substrata; but such a foundation is the safer, and the imposed works are the more surely preserved, if ample automatic tell-tale drainage is provided.

The inclusion of greenheart timbers on the bottom of the gates as contacts between similar timbers on the miter-sills appears to be good design. Probably these timbers would not have been affected by marine borers if the gates and miter-sills had been protected by a flow of fresh water. This could have been accomplished by using small conduits in the side-walls of the locks, conveying fresh water from the upper pool and discharging it continuously along the lower guard-gate and its miter-sill.

The method of replacing the timbers is especially commendable under the existing circumstances, necessitating the practical re-design and installation of contact members between the bottom of these particular gates and their respective miter-sills. It is a question, whether the remedy can be considered better than the original installation of greenheart timbers, provided the timbers had been protected against the activities of marine borers. To have installed the re-designed contact members without placing the lock out of commission, is a notable feat. It illustrates impressively the ingenuity of man when put to the test.

The emergency dams installed subsequent to the original construction were wise provisions against accident. Some such device was undoubtedly in contemplation at the time of the original design of the locks. The use of wickets and girders stored on the lock walls, to be placed in position in the lock by a stiff-leg derrick in an emergency, effects a considerable saving over other forms of emergency dams, but, in view of the importance of these works, the writer believes that the comparatively quicker and more positive balanced cantilever turn-table type, having wickets ready to be lowered into place, would have been preferable. The value of the varied commercial structures damaged by undue delay would justify the additional installation cost and carrying charges. In an emergency, moments of time conceivably may cause hundreds of thousands of dollars of property damage. The best and most rapid type of emergency dam is then none too good or too expensive. It is at such times that the difference in cost is entirely lost to sight and the engineer is hardly thanked for any saving.

Possibly by enlarging the design described practically all the sea water entering Salmon Bay through the locks might be trapped and conveyed back to tide-water. The creation of a salt-water basin above the locks to the greatest reasonable depth would probably be much more effective in trapping the salt water than would result if the basin was only increased in width.

The authors' praise of this fresh-water harbor of 25 000 acres is well justified. It is a signally remarkable harbor. The writer is of the opinion that these works afford harbor facilities that are unsurpassed in the world. The authors are certainly justified in their summarization of the beneficial results to commerce by virtue of the construction of these waterways.

CHARLES E. FOWLER,* M. AM. SOC. C. E. (by letter).†—The writer became interested in the Harbor of Seattle on his arrival there in 1900, when the city had a population of only 85 000, and the harbor facilities consisted of a limited number of pile and timber wharves located on the immediate deep-water front on Elliott Bay. Contracts were soon entered into, by the Company of which the writer was President and Chief Engineer, for the excavation of the East and West Waterways at the head of Elliott Bay and the filling of the eastern portion of the tide flats, which comprised about 1 200 acres. Only 75 acres of this fill had been completed in the five years prior to 1900.

The Waterways were dredged to a width of 1 000 ft. with pier-head lines 500 ft. apart, and a depth of 38 ft. at low tide. The permanent bulkheads were of fir brush, from 12 to 24 ft. in length, laid up to a 45° slope, with the butts outward as the filling progressed. The temporary bulkheads were of piles and plank, of the stepped type, and were used for bringing sections to the grade of 2 ft. above high tide, so that certificates could be issued by the State of Washington for completed work. These certificates drew 8% interest and were a first lien on the land so created. The cost allowed by the State was 16 cents per cu. yd., with an added general profit of 15%, or a total of 18.4 cents per cu. yd. The first contracts up to 1905 were for a total of about 12 000 000 cu. yd., which was excavated and pumped to a distance as great as 5 300 ft. by two 20-in. hydraulic dredges.

As a Trustee of the Seattle Chamber of Commerce and a member of its Harbor Committee, the writer became interested in the promotion of the Lake Washington Canal, and, later, in the formation of the Port of Seattle. One of the first efforts was to convince the War Department and Congress of the necessity, or at least the desirability, of the Canal, and that in place of a \$7 000 000 two-lock canal, a one-lock canal was all that was required. It is interesting to note that the cost estimate made at that time by the writer, with \$3 800 000, and this type of project was finally adopted. Construction was begun by dredging the entrance from deep water in Puget Sound to a place near the present lock site in Salmon Bay. This contract was secured by the writer's Company at 26.7 cents per cu. yd., and much of the material was of the hardest type of blue clay hardpan. The dredge built for the work was of the dipper type, with machinery for a 7-yd. dipper, but owing to the hard material to be dug, a 3½-yd. dipper was used.

The second contract entered into was for the test borings over about 2 miles of the canal route, up to the Ballard Bridge. This was carried out by using a floating pile-driver rig, on the gins of which was placed a square drill shaft turned by a pulley at the deck level, through which the drill bar passed loosely in a square hole. The drills were 2 to 2½-in. augers. The cost of the work under this contract was about 50 cents per vertical ft. The third contract, for additional excavation through Salmon Bay, was

* Cons. Engr., New York, N. Y.

† Received by the Secretary, August 23, 1927.

carried out by the use of the dipper dredge at a cost of about 43 cents per cu. yd.

The construction of the lock* was carried out in a manner that calls for the highest praise of Col. J. B. Cavanaugh and the authors, and also reflects the greatest of credit upon the U. S. Corps of Engineers. The extremely low unit cost of the concrete was due to some extent to the low price obtained on cement at that time, and to the fact that the finest kind of sand and gravel was purchased at 25 cents per cu. yd. on the Government scows at the Point Defiance gravel pits, and towed a distance of about 30 miles to the lock site by the Government tug, *Gen. J. M. Wilson.* Notwithstanding all this, a record was probably set for all time in the matter of cost for such a great piece of work.

The writer also had charge of the final dredging of the Canal through Salmon Bay, Lake Union, and Union Bay in Lake Washington. This work was undertaken at a very low price, less than 9 cents per cu. yd., so that the only way possible of reaching profitable figures was to sell the dirt for filling low lands along the line of the Canal. All that excavated in Salmon Bay was so used, although much of it came out of the 22-in. pipe so soupy as to cause a great deal of it to be lost. The material in Lake Union was sold for a large fill, but much of this was so soft as to make it impossible to bring all the fill to grade. The excavation in Union Bay was in soft silt and peat, which was pumped on to Park property, a distance of 4 000 ft., with an output of as high as 47 000 cu. yd. per day. The digging of the old logging canal to full size, between Lake Union and Union Bay, required the excavation of a large quantity of very tough material (hardpan and cemented gravel), which was as hard as soft concrete. This digging was done by using a rock cutter on the 22-in. electric hydraulic dredge, and pumping the material a distance of about 2 000 ft.

The whole work was most difficult in many respects, yet it was well worth while as one feature of the opening of the last frontier, and was a great step in the building of the Metropolis of the Northwest.†

* *Proceedings,* Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1233.

† Many of the details have been omitted, as they can be found in the writer's book entitled "Sub-Aqueous Foundations."

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE WORK OF THE PORT OF SEATTLE, WASHINGTON

Discussion*

By Messrs. D. W. McMorris, Joseph M. Clapp, and Charles E. Fowler.

D. W. McMorris,† M. Am. Soc. C. E.—There was a very interesting incident in connection with the development of the Port of Seattle, that Mr. Cotterill has not touched upon; the so-called "Bush Terminal Plan of Improvement", that was developed by a number of persons who desired to obtain control of practically all port facilities.

There was a strong sentiment among the more progressive and aggressive members of the community to provide such ample and convenient facilities for trans-oceanic shipments of freight and merchandise as to care for not only all business that would probably reach the port, but also to offer special inducements to new business seeking a Western shipping terminal.

Much was said about the type of development that had been in successful operation at Brooklyn, N. Y., by the Bush Terminal Dock and Warehouse Company.

The report on the "Plan of Seattle" had been recently made by the late Virgil G. Bogue, M. Am. Soc. C. E., with whom the speaker had been associated, laying out a similar plan of development on Harbor Island. "The Harbor Island Terminal Company" was formed and a tentative agreement prepared setting forth with considerable detail the plan of development and the terms of a proposed lease of all the Port's property on Harbor Island.

The effect of the proposed agreement was to have the Port provide the funds necessary to carry on an elaborate construction program and then lease the entire Harbor Island project to the Terminal Company. The sentiment in favor of the project was so strong that it was seriously proposed by some that the Commissioners' regular program give way to the Company's plan

* This discussion (of the paper by George F. Cotterill, Esq., presented at the meeting of the Waterways Division, Seattle, Wash., July 15, 1926, and published in August, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Asst. City Engr., Seattle, Wash.

which would have made the Port merely a means of financing the promoter's project.

Finally, the alluring possibilities of the Harbor Island plan were set aside and the Port development proceeded on more conservative lines. The speaker has always felt disappointed, however, that some way was not found to modify the proposed agreement with the Harbor Island Terminal Company so as to have justified the Port Commission in having had the development completed, as it is certainly an ideal location for such an improvement.

JOSEPH M. CLAPP,* M. AM. SOC. C. E.—In his interesting and valuable paper Mr. Cotterill may have left the impression that the Port of Seattle owned all the water-front area and had developed part of it at a cost of about \$10 000 000 which, together with certain areas deeded the Port from the State, made the value of the Port property about \$12 000 000. The citizens of the District were not unanimously favorable to the development as at present accomplished.

Seattle Port is one of the land-locked harbors of Puget Sound on which a large commercial city is located. There are five such harbors or ports, all located on the mainland and at the mouths of rivers draining into Puget Sound. Puget Sound has a coast line of approximately 2 000 miles composed of a ribbon of sand, flanked on one side by high bluffs and on the other by deep water. Six important rivers empty into the Sound and drain important valleys which constitute the gateways from the back country. Here are found extensive flat areas on which the cities are built.

The City of Bellingham is located near the mouth of the Nooksack River; Everett, at the mouth of the Snohomish River; Seattle, at the mouth of the Duwamish River; Tacoma at the mouth of the Puyallup River; and Olympia, at the mouth of the Deschutes River. No city is built at the mouth of the Nisqually River.

At the mouth of the Duwamish River the flatlands of Seattle were reclaimed, and a large industrial area was created with harbor and waterway frontage provided, all at the south end of the harbor, perfectly landlocked from prevailing storm winds, except an occasional "Northwester". It is here that many of the citizens thought the development of the Port should have been concentrated and ocean piers constructed.

These piers would naturally have followed very closely in design the piers built at Smith's Cove and at the East Waterway. The adjacent areas were available for small and large factories, and this arrangement would have lessened the congestion of freight shipments, now experienced.

Building the piers in this flat area would have lessened the overhead maintenance and left the immediate city water-front for the mosquito fleet and coasting vessels. Many other factors of economy would have resulted. Three of the four transcontinental lines enter Seattle from the south and one from the north. Thus, the location of the piers in the south end of the harbor would have resulted in a single line of railway to handle freight at this point instead of three lines now obliged to handle a large part of their

* Cons. and Contr. Engr., Seattle, Wash.

ocean freight to the north and less protected area. The movement of freight along the water-front from south to north, as at present, creates considerable confusion which is intensified when ocean vessels dock at the central front.

While the Port owns and operates the twelve various developments they are not operated at a profit, but at a loss. All this loss is borne by taxing the property lying within the district. The taxpayer derives some benefit, of course, because of the increased shipping with modern and adequate facilities for handling it. This district also comprises water-front property, privately owned, which is taxed to support a competitor.

CHARLES E. FOWLER,* M. AM. SOC. C. E. (by letter).†—The very considerable part played by the writer in the formation of the Port of Seattle, causes him to read with much interest of the complete success of this municipal undertaking. As a member of the Harbor Committee of the Seattle Chamber of Commerce, the suggestion was made in 1911 of forming an unofficial Seattle Harbor Commission. About fourteen organizations appointed members, and these with the Mayor, the City Engineer and the Corporation Counsel composed the body which planned the Port of Seattle, and secured the passage of the State Law authorizing such legal municipal bodies for any of the ports of the State, comprising either the entire county in which the port was located, or such portion of the county as might be set apart for the district which was to be subjected to taxation for creating the port properties.

The Committee that drew the law, consisted of Corporation Counsel Scott Calhoun, the late A. O. Powell, M. Am. Soc. C. E., and the writer, who believed the district should be greater than the city and less in size than the county, in order to keep it out of local politics. When the Port was organized, it was as the entire County of King, and notwithstanding this, it is pleasant to record that most of the members during the past fifteen years, have been free from political influence. The writer, as Acting Chairman of the Engineering Committee of the unofficial Commission, during most of the formative period, gathered information from all over the world, and made trips to Portland, Ore., San Francisco and Los Angeles, Calif., Chicago, Ill., New York, N. Y., Baltimore, Md., and Washington, D. C., in search of data as to methods of organization and construction in use in the various ports.

This Committee also made plans and estimates of cost of sea walls, piers, warehouses, and all the appurtenances of a port, and it was upon these data that the first \$6 500 000 was fixed for the initial units. The Committee favored one large concentrated terminal, preferably at Harbor Island in Elliott Bay. The first official Commission of the Port of Seattle was very fortunate in having as a member the late H. M. Chittenden, Brig.-Gen., U. S. A. (*Retired*), M. Am. Soc. C. E., and he as first Chairman was also in favor of a concentrated terminal. However, local politics did have a say and, as a result, part of the port improvements are on the East Waterway,

* Cons. Engr., New York, N. Y.

† Received by the Secretary, August 23, 1927.

part on the old water-front of the city at Bell Street, and part on Salmon Bay, or on the Lake Washington Canal. Later, the great concentrated terminal at Smith's Cove was begun, and the full development of this has dwarfed the smaller scattered units, which now only add to the cost of administration.

The smaller units, however, serve the separate districts well in providing space for the vessels in the Puget Sound, coastwise, and Alaskan trade, and the fishing fleet. While the business of the port proper is not a great factor in the growth of a port city, it is likely that these small units are more effective in increasing local business, than is the large terminal at Smith's Cove, where exchange is made from the ocean ships to the rail lines. The City of Seattle should be able to answer much better than the older ports, just how much the meeting of ship and rail lines has to do with making of a metropolis, but in so doing full value must be given to the lumber and Alaskan trade, and to the business carried by the great fleet of smaller vessels plying the waters of Puget Sound.

This is work for the statistician and the port engineer that will be of great value in the future development of coast cities, and in the furtherance of the commerce which passes through them. Certain it is that the great ocean commerce of the future will be found on the Pacific and this will add in untold measure to the land commerce west of the Mississippi River.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

CHARLES IRVING ANDERSON, M. Am. Soc. C. E.*

DIED JUNE 15, 1927.

Charles Irving Anderson was born on July 27, 1878, at Fahlun, Sweden. While still a child he came to America with his parents, who settled in Georgetown, Colo., where Mr. Anderson attended the public schools. He received his technical training at the University of Colorado, at Boulder, Colo., from which he was graduated in May, 1905, with the degree of Bachelor of Science in Civil Engineering.

Mr. Anderson's experience in engineering work, as exemplified in mining and tunneling and the use of mining tools, began prior to the time he entered the University of Colorado. As early as 1898, he was engaged in mining operations and contracted for tunneling work, which contracts he fulfilled both before entering college and during his vacation periods as a student.

Immediately after his graduation in June, 1905, he took charge of a field party in the service of the U. S. Coast and Geodetic Survey. He continued in this work until he entered the Engineering Department of the Illinois Central Railroad System in November of that year, as Masonry Inspector on concrete bridge and retaining wall construction.

In June, 1909, Mr. Anderson left the employ of the Illinois Central Railroad Company to become Superintendent of Construction for shops then being built by George B. Swift Company, at Macon, Ga., for the Central of Georgia Railway Company, and, in March, 1910, he again changed employers to act as General Foreman for the Bates & Rogers Construction Company on the building of subways, retaining walls, conduits, and signal bridges in connection with its contract on the Passenger Terminal of the Chicago & North Western Railway Company, in Chicago, Ill.

In December, 1910, Mr. Anderson returned to the Illinois Central System and was engaged in engineering work with that Company until his death, with the exception of a short period from March 11 to September 30, 1918, when he was given a leave of absence to assist in the construction of the U. S. Government Explosives Plant C, at Nitro, W. Va.

His duties with the Illinois Central Railroad Company were in connection with construction, valuation, and special work relating to ordinances and other matters, sometimes of a technical and at other times of a diplomatic nature. His judgment in matters assigned to him for handling and solution was usually approved by his superiors without question.

At Nitro, his opinions were particularly sought, especially as to foundation conditions, although he was also in charge of roads, drainage, and building construction, subject to orders from the Director.

* Memoir prepared by W. C. Pauli, Assoc. M. Am. Soc. C. E.

In the winter and early spring of 1926, Mr. Anderson's health began to fail quite noticeably, and it was thought that a change of climate might be of some benefit. Accordingly, in June, 1926, he was transferred, at his own request, from the position of Assistant Engineer of Buildings in Chicago to that of Assistant Engineer in charge of track elevation and new passenger facilities at Clarksdale, Miss., and he was so engaged when he was stricken with the illness which culminated in his death. This work, his final assignment, was practically completed at that time.

Mr. Anderson had the happy faculty of making loyal friends, but he also gave loyalty and friendship in prodigal fashion. He seemed to see the best side of a person's character, and was always ready to defend those he knew against attacks reflecting on their ability or integrity.

His life was marked by devotion to his family. Mrs. Anderson survives him, with six children, three sons and three daughters.

He was a member of the Tau Beta Pi Fraternity, Georgetown, Lodge No. 12, A. F. and A. M., Georgetown, Colo., Western Society of Engineers, Structural Engineers Association of Illinois, and the American Railway Engineering Association.

Mr. Anderson was elected a Member of the American Society of Civil Engineers on January 13, 1919.

CHARLES WALTER BRYAN, M. Am. Soc. C. E.*

Died June 25, 1927.

Charles Walter Bryan was born in Warren County, Missouri, on May 5, 1863. He was the second son of Archibald Bryan and Mary Elizabeth Sterigere, and a lineal descendant of Daniel Boone. The Bryan family came to Missouri about 1800 and were prominently identified in the early settlement of Kentucky and Missouri.

From 1869 to 1875, he attended private schools, and from 1875 to 1880 the public schools of Washington, Mo. During the summers from 1878 to 1880, Mr. Bryan worked on his father's steamboats, which carried supplies to the Upper Missouri and Yellowstone Rivers. During these trips as an Assistant Pilot he saw many phases of frontier life. In the fall of 1880 he entered Washington University, St. Louis, Mo., and was graduated therefrom with honor as a Civil Engineer in 1884. In 1905, Washington University conferred upon him the honorary degree of Master of Arts.

From July to November, 1884, Mr. Bryan was a Draftsman with the Pond Engineering Company, St. Louis. In November, 1884, he was called to the Edge Moor Iron Company, of Wilmington, Del., as a Draftsman by George H. Pegram, Past-President, Am. Soc. C. E., Chief Engineer. From November, 1885, to June, 1886, he was a Draftsman with the noted bridge engineer, the late C. Shaler Smith, M. Am. Soc. C. E., at St. Louis, and from June, 1886, to June, 1887, he was again a Draftsman with the Edge Moor Iron

* Memoir prepared by Otis E. Hovey and C. G. Emil Larsson, Members, Am. Soc. C. E., and George H. Pegram, Past-President, Am. Soc. C. E.

Company. From June to September, 1887, he was in the Bridge Department of the Missouri Pacific Railway, and, in September, 1887, he returned to the Edge Moor Iron Company as Engineer in Charge of design and construction. During 1888, the Edge Moor Bridge Works was organized and Mr. Bryan continued his work with the new Company. He was appointed Chief Engineer in February, 1896, and retained this position until the Edge Moor Bridge Works was acquired by the American Bridge Company during 1900. At Edge Moor, he designed and had charge of the construction of the Wheeling Bridge, Wheeling Bridge and Terminal Railway; the Steubenville Bridge and Cincinnati Bridge, Pennsylvania Lines West; the Kenova Bridge, Norfolk and Western Railway; the Cincinnati Bridge, Cincinnati and Covington Suspension Bridge Company, and many other important structures.

During the Edge Moor period, Mr. Bryan developed unusual ability as a salesman, in addition to his skill in purely engineering work. The American Bridge Company appointed him as Agent, in the Railway Contracting Department, at Pittsburgh, Pa., and during 1901 he became Division Contracting Manager of the American Bridge Company of New York, at New York, N. Y. During 1906, he was appointed Chief Engineer of the American Bridge Company of New York, in addition to his duties as Division Contracting Manager. Early in 1911, he was relieved of his duties as Division Contracting Manager and appointed Chief Engineer of the American Bridge Company, which position he held at the time of his death. In this position Mr. Bryan was the executive and responsible head of the engineering organization of a large corporation engaged in the construction of bridges, buildings, and other structures in the United States and in many other countries. In addition to his engineering duties, he was frequently called upon for advice in matters relating to contracting.

Mr. Bryan was joint author with the late J. B. Johnson and F. E. Turneaure, Members, Am. Soc. C. E., of the well-known treatise, "Modern Framed Structures", first published in 1893.

He was one of the founders of the Huguenot Trust Company, in New Rochelle, N. Y., of which he was a Director and served as President from 1912 to 1915, and afterward as Chairman of the Board of Directors. He had also served on the Board of Education, the Board of Public Works, and the Park Commission of New Rochelle. He was one of the organizers of St. Paul's Protestant Episcopal Church, a Vestryman, and Chairman of the Building Committee.

"If a man die shall he live again?" It is generally believed that he will, and all agree that a man of such fine character as Mr. Bryan does live in his permanent influence for good. He was a gentle, modest, and friendly man whose qualities endeared him to all his friends and associates. He was considerate of the feelings and opinions of others, but had definite ideas of his own, and when it was necessary to insist upon them, he had a way of quietly leading one to his point of view in a shrewd and effective manner which left no sting of disappointment. His mind was keen and active, and he seemed to sense a situation almost by instinct. He had unusual ability to see another man's point of view, and it often seemed as if he penetrated hidden motives

by intuition. He was gifted with a quaint and sparkling humor which made even a business interview with him a pleasure to be remembered. He had a genius for friendship in both business and social relationships. His friends were friends to the end, and there are many who will sadly miss the intimate associations of years.

In his earlier experience Mr. Bryan was largely occupied in the design of bridges. He had excellent technical ability and was accurate and very rapid in his work. His designs were marked by simplicity of construction and were well adapted to their economic use. In the later period, while Chief Engineer of the American Bridge Company, his executive responsibilities largely prevented him from undertaking actual designs. His work was rather to guide those associated with him toward better design and better methods of construction in the great number and variety of structures built by the Company. He was freely consulted by members of the Engineering Staff and by customers, their engineers, and others, and in this way his influence toward better engineering was greater and more widely felt than would have been possible if he had been engaged in private practice. He always sought for the best, and kept fully abreast and often in advance of the progress of engineering practice.

With his attainments as an engineer Mr. Bryan combined good business ability, and for many years his work was of a kind in which each was of equal importance. He quickly established cordial relations with customers, and they became and continued to be his friends.

Mr. Bryan's personal qualities, his ability, and his keenness of mind were such that his influence on his associates was strong, and was particularly stimulating to younger men, in whom he was much interested. Many applied to him for advice and guidance, and he gave freely of his time and thought in their behalf.

Mr. Bryan maintained a growing interest in his Alma Mater, and did much for its advancement. He was one of the founders of the Washington University Alumni Association in New York, and frequently served as its President.

He was a good citizen and, as such, was always interested in public questions and gave liberally of his time whenever he found a way to be of service to the communities in which he lived. His public service was unselfish and for the good of all and never for his own advancement.

One cannot write the sum of the influence of the life of such a man as Mr. Bryan; but one can be sure that it is great and will continue. The members of the Engineering Profession are fortunate to have been able to count him among their number, and will remember him as another of the great men who have finished their work and left a record which will be an inspiration to those who follow.

In 1889, Mr. Bryan was married to Mary Elizabeth Shaw, daughter of David and Ann Poole Shaw, of Philadelphia Pike, near Edge Moor, Del. He is survived by his wife and by his four children, Charles W. Bryan, Jr., of New York, Philip D. Bryan, of Albany, N. Y., Mrs. William F. Washburn, of Rochester, N. Y., and Mrs. Lowell Milligan, of Worcester, Mass.

He was a Member of the American Iron and Steel Institute, the American Society for Testing Materials, and the St. Louis Engineers' Club. He was also a member of the New Rochelle Yacht Club, the Wykagyl Country Club, the Engineers' Club, and the Railroad Club in New York.

Mr. Bryan was elected a Member of the American Society of Civil Engineers on June 3, 1903.

CHARLES GOBRECHT DARRACH, M. Am. Soc. C. E.*

Died June 1, 1927.

Charles Gobrecht Darrach, the son of William and Christiana Elizabeth (Gobrecht) Darrach, was born in Philadelphia, Pa., on September 8, 1846, and was of distinguished Colonial ancestry.

Mr. Darrach was graduated from the Philadelphia Central High School in 1866, and began his technical career as Assistant Engineer in the office of George W. Leuffer, Chief Engineer of the Philadelphia and North Branch Railway survey, with whom he remained until 1867.

He held successively the following positions: In 1868, he was Assistant Engineer with the Reading Railroad Company and, during 1869, with the Jersey Shore, Pine Creek, and Buffalo Railroad Company. From 1870 to 1871, he served as Principal Assistant Engineer with the New Orleans and Selma Railroad Company; and, in 1871, he was employed as Locating Engineer with the Baltimore and Ohio Railroad, from Tiffin, Ohio, to Chicago, Ill. During 1872 and 1873, he was Assistant to the Chief Engineer of the Elizabethtown, Lexington, and Big Sandy Railroad Company (now the Chesapeake and Ohio Railroad Company); and from 1873 to 1875, Construction Engineer for the Reading Railroad Company.

In 1875, Mr. Darrach accepted the appointment of Principal Assistant Engineer of the Water Department of the City of Philadelphia, under Dr. William H. McFadden, and remained with the Department until 1885, when he became a member of the firm of Wilson Brothers and Company, Architects and Engineers. His work in this connection consisted in the design and construction of a large number of office buildings and institutions, among which were the Drexel Institute, the Reading Terminal, the United Gas Improvement Building, and the Fidelity Building, in Philadelphia. This construction covered a period of thirteen years. In 1888, Mr. Darrach also designed and constructed the Water-Works System in the Borough of Ridley Park, Delaware County, Pennsylvania.

In 1899, he engaged in private consulting practice which he continued about five years before his death on June 1, 1927, after an illness of several months. He was identified with much important appraisal work for railway and electric traction systems; made expert examinations and reports in legal cases; and prepared many reports, papers, and discussions on railways, public utilities, water and sewage purification, the mechanical installation for office

---

* Memoir prepared by J. W. Ledoux, M. Am. Soc. C. E.

and other large buildings, and the heating and ventilation of hospitals, etc. One of his most important contributions to engineering literature was his paper on "Mechanical Installation in the Modern Office Building".*

He also made reports on proposed improvements of port facilities and the betterment of water supply and sewage disposal in Philadelphia, and planned and constructed the water-works, drainage, and sewerage systems for various towns, for the State Insane Asylum, at Wernersville, Pa., and for power plants, heating and ventilating systems, electric installations, etc., for large buildings in Philadelphia and elsewhere.

Mr. Darrach was a man of sterling integrity and, as a good citizen, was greatly interested in municipal matters, particularly those involving engineering problems. He had very strong religious convictions, which he practiced throughout his life.

He was the founder and first President of the Institute of Operating Engineers, a member of the American Geographical Society, and an Honorary Member of the American Association of Hospital Superintendents. He was also a member of St. James's Protestant Episcopal Church, of Philadelphia.

He was married, in 1876, to Martha Amy Elizabeth Tearne who survives him, together with three daughters, all of whom reside in or near Philadelphia, and two sons, Charles G. Darrach, Jr., of Los Angeles, Calif., and Walter T. Darrach, of Miami, Fla. He was a most affectionate husband and father.

Mr. Darrach was elected a Member of the American Society of Civil Engineers on January 5, 1876.

FRANCIS CLARKE GAMBLE, M. Am. Soc. C. E.†

Died November 13, 1926.

Francis Clarke Gamble, the son of Clarke Gamble, and Harriet Eliza (Boulton) Gamble, was born in Toronto, Ont., Canada, on October 23, 1848. As a boy he was educated in the public schools of that city, after which he attended Upper Canada College, where he received his primary education, later continuing his studies under private tuition. After four years of practice in field engineering on the Intercolonial Railway and the Great Northern Railway of Canada, Mr. Gamble entered Rensselaer Polytechnic Institute, Troy, N. Y., where for a year or more he took a special course. It was there that the writer first met him and, being fellow countrymen, a close friendship soon developed, that endured for more than half a century.

At Rensselaer, Mr. Gamble, being a special student, thus belonging to no particular class, and being several years older than most of the other students, did not become widely acquainted; but he was well liked by all who knew him intimately. In selecting his studies he chose those that were mainly of a utilitarian character, especially those that bore on his adopted specialty of

* *Transactions*, Am. Soc. C. E., Vol. XLVIII (1902), p. 1.

† Memoir prepared by J. A. L. Waddell, M. Am. Soc. C. E.

railroading. That he probably made a wise selection is shown by the success he achieved in after life; for not only did he hold high positions in railroading in Canada, but he was also elected President of the Canadian Society of Civil Engineers (now the Engineering Institute of Canada).

After leaving Rensselaer, Mr. Gamble worked for a short time as Contractor's Engineer on the Prince Edward Island Railway, then returned to the Intercolonial Railway Company as Engineer in charge of ballasting. His next position was as Transitman on the Georgian Bay Branch of the Canadian Pacific Railway, and late in 1877, he served as Assistant Engineer of Construction on the Quebec, Montreal, and Ottawa Railway. He then returned to the Canadian Pacific Railway Company as First Assistant Engineer on Contracts Nos. 42 and 62.

In the spring of 1881, he entered the Department of Public Works of the Dominion Government in British Columbia; and, in 1889, he became Resident Engineer in that Department, having in charge all harbor and river improvement and the erection of public buildings. It was not long before he was placed in full charge of all public works under the Dominion Government in the Province of British Columbia.

In 1897, Mr. Gamble was engaged in private practice; and in 1898, he was appointed Public Works Engineer for the Provincial Government of British Columbia, becoming, in 1910, Chief Engineer of the Department of Railways for the Province, which position he held until 1918, when he retired from active practice. He continued to reside in Victoria until his death.

He took a great interest in the affairs of the Engineering Institute of Canada, becoming a member in January, 1887. He was elected a Member of Council in 1892 and again in 1898. In 1913 and 1914, he was Vice-President, and, in 1915, he became President. He was also a member of the Institution of Civil Engineers of Great Britain.

Within the profession and in all walks of life Mr. Gamble was held in the highest esteem by all with whom he came in contact. He was a man of the strictest integrity, and was always governed in his actions by the loftiest ethical principles and ideals. He was recognized throughout the Dominion as a high authority on the various professional works to which his energies were devoted.

He is survived by his only son, Clarke W. Gamble, of Cairo, Ill.

Mr. Gamble was elected a member of the American Society of Civil Engineers on April 1, 1891.

BENJAMIN FRANKLIN HOWLAND, M. Am. Soc. C. E.*

Died May 27, 1927.

Benjamin Franklin Howland, the son of Reuben R. and Martha Y. (Brightman) Howland, was born at New Bedford, Mass., on June 8, 1877.

* Memoir prepared by Sidney G. Walker, Esq., Honolulu, Hawaii.

His early years were spent in his native city, where he received his primary and secondary education. He later attended the Friends' School, at Providence, R. I.

After a few years of business experience in Boston, Mass., Mr. Howland removed to Hawaii in 1899, entering the employ of a firm of civil engineers and surveyors at Hilo, and for several years was engaged in a variety of work having to do with railroad location, underground water development, and intricate surveys. In 1903, he became the Engineer for a large sugar company, and, in 1905, was employed, at Honolulu, by the Engineering Bureau of the Public Works Department for the Territory. In this position he was responsible for the design of extensions to the sewerage system, roads, bridges, reservoirs, and City water-works, also acting at times as engineer for the City and County, with its numerous and varied demands.

In 1909, he spent a number of months in Cuba, as Engineer for a large sugar corporation, and after his return to Hawaii in 1910, was employed in several responsible capacities, until he left to take a special course at the Massachusetts Institute of Technology, where he spent most of 1912 and 1913.

For several years, following his return to Honolulu, in 1914, Mr. Howland was employed by a prominent contracting company in designing, estimating, and as Superintendent of Construction, on numerous important contracts, several of which were for buildings of the United States Government.

In 1920, he became Chief Engineer for one of the large sugar agencies, controlling thirteen important plantations, in which capacity he designed and erected sea walls, dams and reservoirs, sewerage systems, water supply and power pipe lines, together with numerous buildings and residences.

At various times, he was employed by the Territorial Survey Department, as an expert in adjusting old surveys. In the later years of his life, he was a partner in an engineering organization, specializing in the design and installation of fire protective apparatus for safeguarding large concentrations of value in and about the sugar-mills and pineapple canneries of the various Islands in the group.

Mr. Howland's alert mentality, unswerving integrity, and tireless energy, combined with a long and varied experience in the multitude of engineering problems of a rapidly growing city, and in the development and operation of the hydraulic and transportation elements of large-scale sugar production, made his services and advice of inestimable worth in the community, to the welfare of which he devoted so large a share of his professional career. He was a member of the Benevolent and Protective Order of Elks.

On November 24, 1914, he was married to Rheta C. Macdonald, of Boston, Mass., and to this union were born two daughters, Rheta Russell, and Anne Bartlett Howland, all of whom survive him.

Possessing rare qualities of heart, his home life was exceptional, and his passing has bereft a large circle of friends and intimates which he had built around a useful and unselfish life.

Mr. Howland was elected a Member of the American Society of Civil Engineers on October 2, 1922.

CHARLES DAVIS JAMESON, M. Am. Soc. C. E.*

Died February 13, 1927.

Charles Davis Jameson was born in Bangor, Me., on July 2, 1855, the eldest son of Charles Davis Jameson, who during the Civil War served as a General in the Union Army, and Julia Lambert (Smith) Jameson. He passed his boyhood chiefly in Maine, attending school in Stillwater, Bath, and Bangor, principally at Dillingham's School at the latter place. After the death of his father, he lived for some years in Memphis, Tenn., with his uncle, Judge Thomas R. Smith. Later, he attended Bowdoin College at Brunswick, Me., where he was a member of the Alpha Delta Phi Fraternity, and from which he was graduated in Civil Engineering in 1876.

Mr. Jameson's first work was on the Bangor Dam across the Penobscot River, following which he went to St. John, N. B., Canada. In 1878, he became Section Engineer on the Memphis and Charleston (now the Southern) Railroad from Memphis, Tenn., to Huntsville, Ala.

Two years later he accepted a position in Mexico as Engineer in Charge and Superintendent of Construction on the Mexican Central Railroad. He was first stationed in the City of Mexico, but, later, was transferred to the sections from Leon to Irapuato. He also built tramways in Leon and Guanajuato, and was engaged in the preliminary location from Silao to Lake Chapala and Guadalajara, and from Tepic to San Blas.

In 1883, Mr. Jameson went to the Isthmus of Panama where he was employed under Count de Lesseps, as Assistant Engineer of the American Contracting and Dredging Company, and, later, as Chief Engineer and Superintendent. Having contracted the Chagres fever, he was forced to return to the United States. In 1885, he made standard plans for wooden structures for the European and North American Railroad Company.

On June 23, 1886, he was married to Florence Miller, of New York, and Memphis, Tenn. During the winter of 1886-87 he taught Engineering at the Massachusetts Institute of Technology, Boston, Mass.; and from 1887 to 1895, served as Head of the Department of Engineering at Iowa State University, Iowa City, Iowa.

In 1895, Mr. Jameson went to North China where for the next twenty-three years he was actively engaged in his profession. He became Chief Consulting Engineer and Architect for the Imperial Chinese Government, and, as such, made many surveying trips throughout China, Manchuria, etc., penetrating to places never before reached by white men. He also built a number of Government buildings, notably the Wai-Wu Pu Foreign Offices, the best constructed Government building in China.

After the fall of the Empire, Mr. Jameson became Chief Engineer for the American Red Cross. In this connection, he drew up all the plans for river conservancy in connection with flood and famine relief in China. These elaborate plans, involving the expenditure of about $50 000 000 by the Chinese

* Memoir prepared from information on file at the Headquarters of the Society.

Government, have been interrupted by the World War and subsequent conditions in China.

In 1918, he returned to the United States and made his home in Bangor. He died in Sarasota, Fla., on February 13, 1927. On January 31, 1927, Mr. Jameson was married to Margaret French, of Washington, D. C., and is survived by his widow, a son, Wylie Blount Jameson, and a grandson, Wylie Blount Miller Jameson.

He was a member of India House, the Cosmos Club, of Washington, D. C., and the Loyal Legion. He was the author of a book on the making of Portland cement, of numerous monographs in engineering journals, and of an account of his experiences during the Boxer troubles in China, which was privately printed.

Mr. Jameson was elected a Member of the American Society of Civil Engineers on March 7, 1888.

FRANK PRICE, Affiliate, Am. Soc. C. E.*

Died February 5, 1927.

Frank Price was born at Dos Oris, Glen Cove, Long Island, N. Y., on May 4, 1852, the son of George James Price, a prominent New York City business man and builder, and Susan Louise (Thompson) Price. He was educated in public and private schools, and by tutors, and gained a thorough mechanical and practical training by five years' service in the structural iron works of Bailey and Debevoise. He continued his practical experience on the Pacific Coast, with vacations spent as cowboy and sheep herder, thereby building up a rugged constitution.

In 1879, Mr. Price was Superintendent of First Order United States Lighthouses at the Phœnix Iron Works, Trenton, N. J. Later he designed and supervised the construction of the machinery and apparatus for moving Cleopatra's Needle, the Egyptian obelisk which was presented to the City of New York in 1880 and erected in Central Park in 1881. It is a granite monolith 70 ft. long, weighing nearly 400 tons. The handling and transportation of such a slender and brittle shaft under the heavy stresses developed by its own weight, as well as the necessity of moving it from a vertical to a horizontal position, presented a difficult and unprecedented problem. Mr. Price accompanied the late Henry H. Gorringe, Commander, U. S. N., Affiliate, Am. Soc. C. E. to Alexandria, Egypt, to take charge of dismounting and shipping the obelisk in a special ship to New York, thence transporting it to Central Park and erecting it.

In 1881 he superintended the making of important repairs to Princeton University Observatory, Princeton, N. J., and, until 1883, was in charge of the Structural Department of the Phœnix Iron Company, at Trenton. From 1883 to 1885, he was Assistant Superintendent, and, later, Assistant to the Receiver of the American Ship Building Company, at Philadelphia, Pa. In

* Memoir prepared by Frank W. Skinner and James B. French, Members, Am. Soc. C. E.

1885, he was Superintendent of the reconstruction of the famous New York Stock Exchange, replacing the roof, 100 ft. high, with massive steel girders, without interrupting business.

In 1888, Mr. Price was Superintendent of the Composite Iron Works, New York, N. Y. In 1889, he founded the Price and Company Structural Iron Works, New York, of which he was chief owner, President, and Director of Designs. After disposing of this business in 1894, he was able to spend more time on the beautiful family estate, Dos Oris, where he carried out many improvements of a structural nature intended to preserve this old Colonial residence, adding modern sanitary and architectural features and enhancing the great natural beauties of the place. This was his most delightful avocation, to which he devoted more and more time, as opportunity afforded, during his entire life.

In 1889, Mr. Price was Superintendent of construction of large reinforced concrete bridges in Porto Rico and, from 1902 to 1908, he acted as Mill and Shop Inspector for the Long Island Railroad Company. In 1908, he was Chief Shop Inspector of Transmission Towers for the Electric Bond and Share Company, New York.

From 1909 to 1911 he held the position of Chief Shop Inspector for structures built for the New York State Barge Canal, and, from 1911 to 1912, he was Chief Shop Inspector, for the United States Government, of the 60 000-ton, steel lock-gates and fender chains for the Panama Canal. In 1915, Mr. Price was retained by the Federal Government as expert witness in the $2 500 000 Panama Canal, lock-gate litigation and from 1915 to 1927, he was engaged on bridge and building work for the Long Island Railroad Company and other organizations.

Mr. Price was a competent engineer, an able designer, and a successful constructor. His skill, experience, and broad knowledge of machinery, materials, and methods, added to his high standards of workmanship and ethics, and his appreciative consideration of employers and employees, made his shop and field work unusually efficient. He was a close observer and a deep thinker, of much ingenuity and originality. He had great mechanical skill and ability in the use of tools, and found recreation in delicate construction, as demonstrated by the miniature model of Cleopatra's Needle, and the mechanical apparatus used for its transportation and re-erection in New York, which he made and presented to the Metropolitan Museum of Art, together with a concise description of the obelisk.

Mr. Price was a student of the best English and American literature, and a charming writer. He loved Nature, and was fond of outdoor occupations and sports, especially boat-building and sailing. His disposition was manly and straightforward, with a vein of quiet humor, and warm affection and loyalty for his many personal and professional friends. He never married, and always made his home in the old family residence. His death was due to a violent attack of pneumonia occurring before he had fully recovered from a similar attack a few months previous.

Mr. Price was elected an Affiliate of the American Society of Civil Engineers on February 6, 1912.